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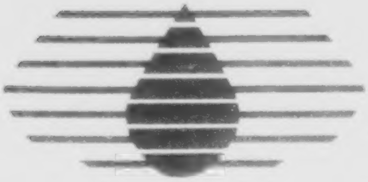

Comptes Rendus
Proceedings

11e Symposium sur le traitement des eaux usées
11th Symposium on wastewater treatment

COMMUNAUTE URBAINE DE MONTREAL

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PRÉFACE

Le comité d'organisation du 10ième Symposium sur le traitement des eaux usées est très heureux de vous souhaiter la plus cordiale des bienvenues à cet événement annuel. Le succès toujours croissant de cette conférence est le résultat de votre support constant durant les dix dernières années et qui a permis d'identifier le symposium comme étant la fine pointe du transfert technologique en matière de traitement des eaux usées au Canada.

De plus, le symposium a acquis une excellente réputation internationale grâce au degré supérieur d'échange scientifique qu'il a réussi à procurer.

Le symposium s'adresse aux chercheurs, ingénieurs, opérateurs, technologues et donc aux spécialistes dans les domaines de la recherche, de la conception de l'opération et de la gestion d'installations de traitement des eaux usées. Le principal objectif du symposium est de promouvoir et de diffuser l'expertise des techniques du traitement des eaux usées domestiques et/ou industrielles.

Des spécialistes de plusieurs pays communiqueront le fruit de leurs expériences, de leurs connaissances et de leurs recherches dans le domaine du traitement des eaux usées.

André Champoux
Ronald Zaloum
Liette Roy
Kamal Karasivan

PREFACE

The organizing committee of the 10th Symposium on Wastewater Treatment is extremely pleased to welcome you to this annual event. The constantly increasing success of this conference is the result of your continued support over the last ten years, and it has become a leading forum for the exchange of wastewater technology in Canada.

In addition, the degree of excellence in Technology transfer that the symposium has provided has gained international recognition.

The symposium is intended for scientists, engineers, technicians plant operators and specialists working in research and the design and the operation of wastewater treatment facilities. The principal objective of this symposium is to promote the exchange of state-of-the-art expertise in the field of municipal and industrial wastewater treatment.

Guest speakers from a variety of specialties and countries will share their most recent experiences and improvements in the wastewater treatment field.

Thérèse Drapeau
Denis Lapointe
Serge Rouleau

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THE USE OF SNOWMAKING FOR WASTEWATER TREATMENT AND DISPOSAL

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INTRODUCTION

Studies of meltwater generated from natural snowpacks have shown that an inordinately large fraction of the impurities present in the pack is concentrated into the early portion of the snowmelt during the spring thaw (Johannessen and Hendriksen, 1978). During the snow formation and after deposition, impurities are forced out of the forming ice crystals and concentrated onto the outer surface of the snow grains and into the interstitial water, from there they are readily removed by a wetting front moving through the snowpack (Colbeck, 1981).

It was reasoned that by taking advantage of this concentration phenomenon, the conversion of secondary effluent into snow could provide a viable alternative method of effluent disposal for communities facing effluent disposal problems during the winter months (Zapf-Gilje et al., 1986). These communities include:

1) Communities practicing seasonal effluent spray irrigation in cold weather climates;
2) Communities with inadequate surface water dilution during the winter months;
3) Ski resorts, which generate maximum flows during the winter months when creek flows are generally too low to allow effluent discharge, and which often have insufficient suitable land for conventional ground disposal.

There are a number of beneficial attributes to the use of snowmaking as a means of effluent disposal. For example, if an effluent were applied as snow onto agricultural or forested land during the winter months, the majority of the nutrients present in the snowpack would be concentrated into the early portion of the snowmelt. The soil would be saturated with this nutrient-rich concentrate, leaving the remaining, relatively pure meltwater as surface runoff. An additional benefit of snowmaking to agriculture and silviculture is that the snow cover provides a considerable measure of protection against frost-kill.

RESEARCH PROGRAM

A pilot-scale research program was undertaken during the winters of 1986/87 and 1987/88 to establish practical design effluent disposal. The study was funded by Supply and Services

Canada, Environment Canada, the City of Kamloops, the British Columbia Ministry of Environment, and the Science Council of British Columbia. Details of the experimental methodology and the equipment used are presented elsewhere (Rabinowitz et al., 1988; Hyslop et al., 1988).

The research program was designed to evaluate the influence of two key design parameters on the impurity concentrating efficiencies, specifically:

1) snowpack height (0.5 to 4.5 metres in depth);
2) snow density (300 to 600 kg/m^3)

The study was carried out at the City of Kamloops effluent spray irrigation site at Cinnamon Ridge, British Columbia. Three meltwater samplers and three soil column lysimeters were installed at the base of each snowpack to collect meltwater during the course of the snowmelt. A total of six snowpacks of varying heights and densities were produced, resulting in a total of eighteen pack height and density combinations. Snow was made from effluent from the City of Kamloops aerated lagoon system using a Snow Warrior "airless" snowgun, manufactured by Turbocristal Inc. of Quebec City.

The meltwater samples collected were analyzed for both nutrient and metal parameters. Table 1 illustrates the impurity concentrations in the lagoon effluent used during the study.

TABLE 1. LAGOON EFFLUENT IMPURITY CONCENTRATIONS

PARAMETER	1986/87	1987/88
TKN (mg N/L)	24.3	24.6
NH_4 (mg N/L)	22.6	22.4
NO_x (mg N/L)	1.8	0.2
TP (mg P/L)	3.5	1.4
PO_4 (mg P/L)	3.1	1.1
Ca (mg/L)	23.8	22.9
Fe (mg/L)	0.05	0.28
Mg (mg/L)	10.0	9.8
Si (mg SiO_2/L)	11.5	11.9
Na (mg/L)	48.9	45.5
Sr (mg/L)	0.15	0.15
K (mg/L)	10.8	11.4
B (mg/L)	0.19	0.19
Mn (mg/L)	0.013	0.045

Of specific interest was the total mass of each impurity released from the snowpack in the first 10, 20 and 30 percent of the meltwater collected, as this fraction contained the highest impurity concentrations.

Typical plots of the cumulative percent mass of nutrients and metals released from the snowpack as a function of the cumulative percent meltwater are shown in Figures 1 and 2, respectively. Figure 1 shows evidence of the concentration phenomenon previously described, illustrating that the degree of nitrogen concentration exceeds the degree of phosphorus concentration. Typically, about 90% of the nitrogen and 70% of the phosphorus was removed from the snowpack in the first 30% of the meltwater. Figure 2 illustrates that the metal constituents exhibited an even wider variation in concentrating efficiency than the nutrients. Magnesium, sodium, potassium and boron all have initial meltwater concentrations similar to that of nitrogen whereas iron and manganese exhibited little or no evidence of enhanced concentration.

A summary of the range of nutrient and metal removal efficiencies observed in the eighteen meltwater samplers is presented in Figure 3.

For each impurity presented in Figure 3, there is a range of observed removal efficiencies. In general, the higher removal efficiencies were achieved with increasing pack height and decreasing snow densities, as indicated by the following simplified monograph:

HIGH SNOW DENSITY		**LOW SNOW DENSITY**
	▌↔↔↔↔↔↔↔↔↔↔↔↔↔↔↔↔▌	
LOW PACK HEIGHT		**HIGH PACK HEIGHT**

INCREASING REMOVAL EFFICIENCY ----->

The variation in the degree concentration between impurities present in the effluent may be attributable to a number of factors, including:

1) impurity association with particulates;
2) chemical properties of the impurity and the extent of rejection from the crystalline lattice;
3) relative concentration of the impurity in the effluent.

Contour plots of the mass of Total Kjeldahl Nitrogen, Nitrate and Total Phosphorus removed from the snowpack in the first 10%, 20% and 30% of the meltwater, as a function of snowpack height and snow density, are presented in Figures 4, 5 and 6, respectively. These plots illustrate the interaction between snowpack height and snow density as they affect nutrient removal efficiencies. For example, at pack heights between 2.0 and 4.5 metres, efficient nitrogen removal was achieved over a wide range of snow densities, whereas for pack heights below 2 metres, the snow density influences the degree of concentration, with efficient removal occurring for snow densities below approximately 400 kg/m^3.

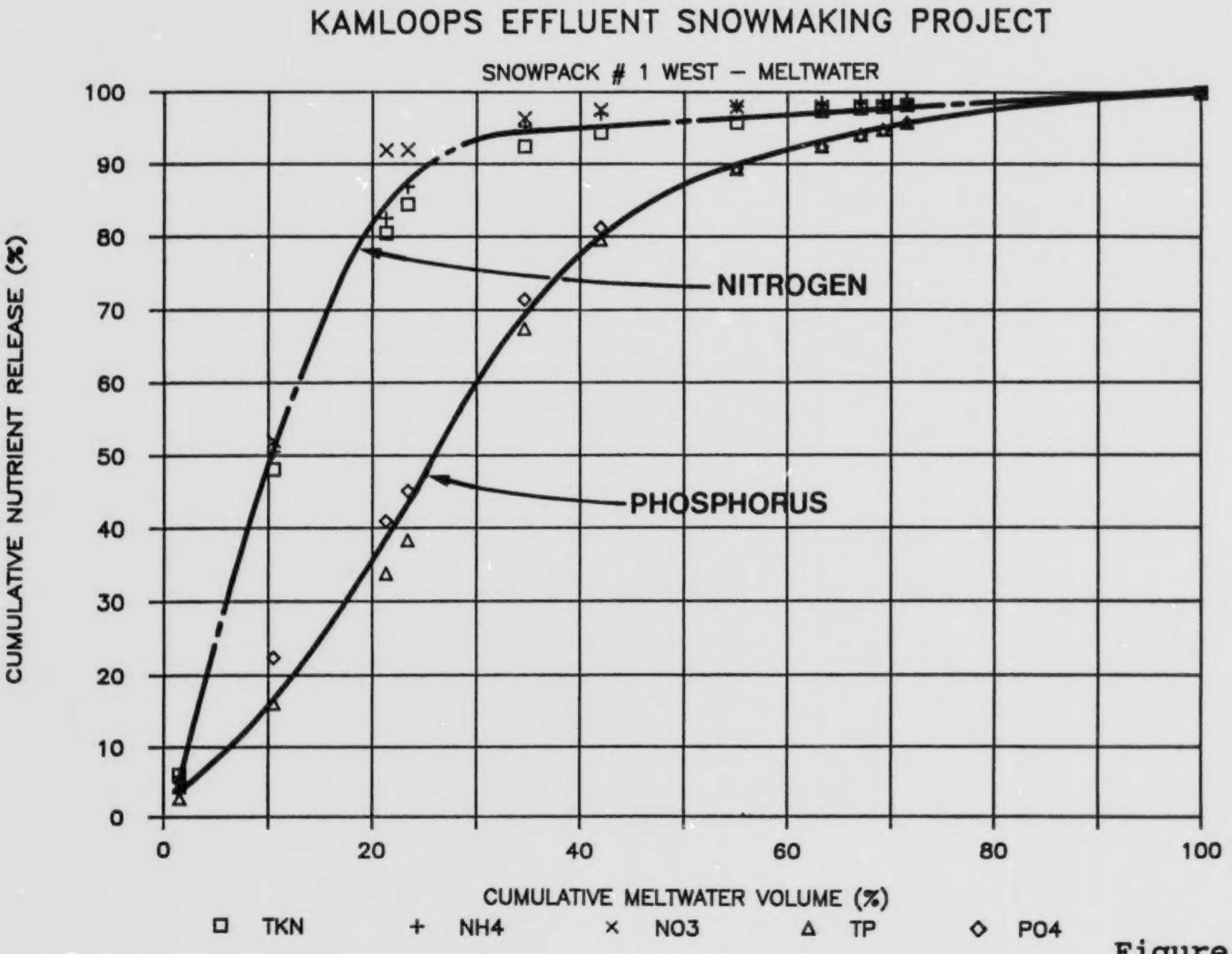

KAMLOOPS EFFLUENT SNOWMAKING PROJECT
SNOWPACK # 1 WEST – MELTWATER
CUMULATIVE NUTRIENT RELEASE (%)
0
10
20
30
40
50
60
70
80
90
100
NITROGEN
PHOSPHORUS
0
20
40
60
80
100
CUMULATIVE MELTWATER VOLUME (%)
□ TKN
+ NH4
× NO3
△ TP
◇ PO4

Figure 1

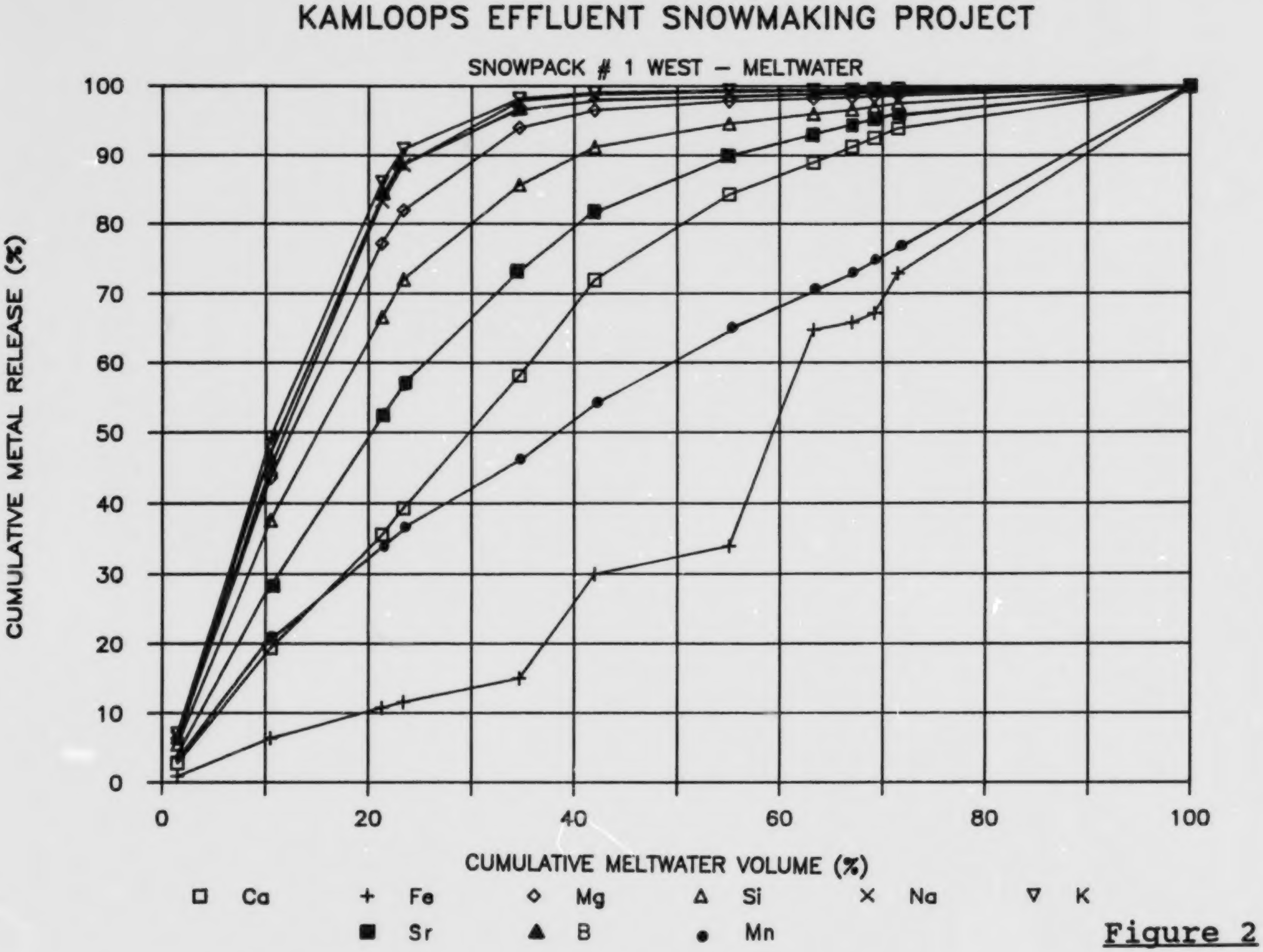
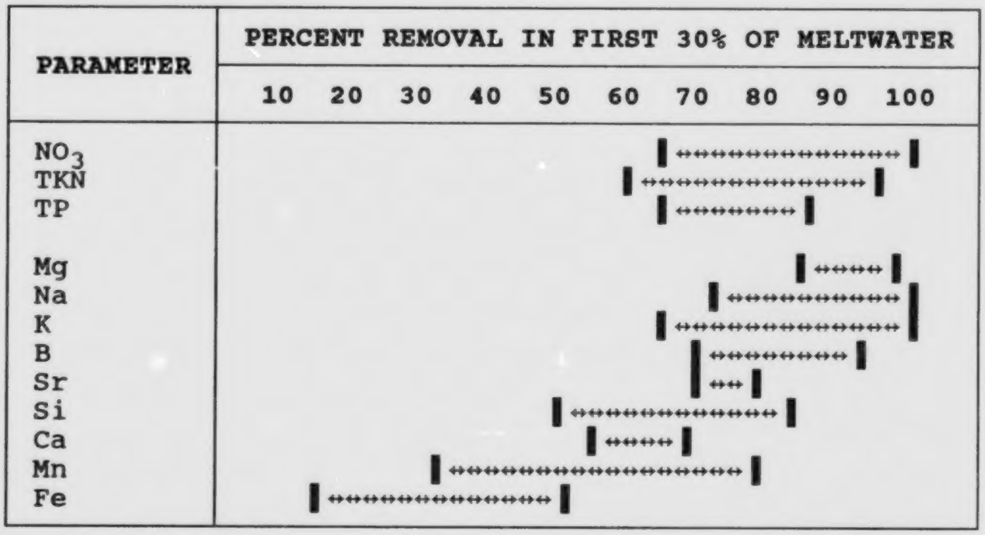

KAMLOOPS EFFLUENT SNOWMAKING PROJECT
SNOWPACK # 1 WEST – MELTWATER
CUMULATIVE METAL RELEASE (%)
100
90
80
70
60
50
40
30
20
10
0
0
20
40
60
80
100
CUMULATIVE MELTWATER VOLUME (%)
□ Ca
+ Fe
◇ Mg
△ Si
× Na
▽ K
■ Sr
▲ B
● Mn

Figure 2

FIGURE 3. OBSERVED IMPURITY REMOVAL EFFICIENCY RANGES



PERCENTAGE OF TOTAL KJELDAHL NITROGEN (TKN-N) REMOVED FROM SNOW PACK

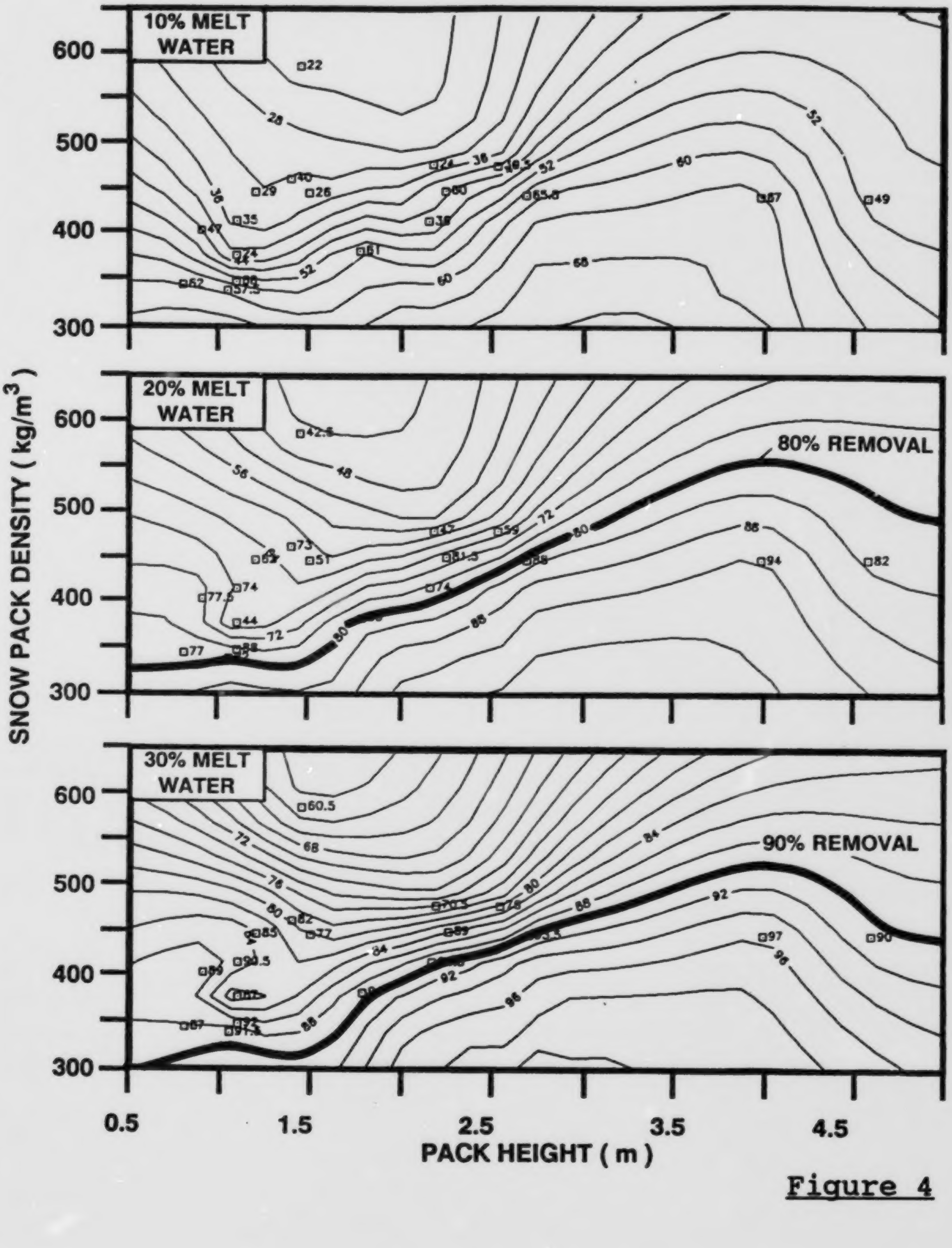


Figure 4

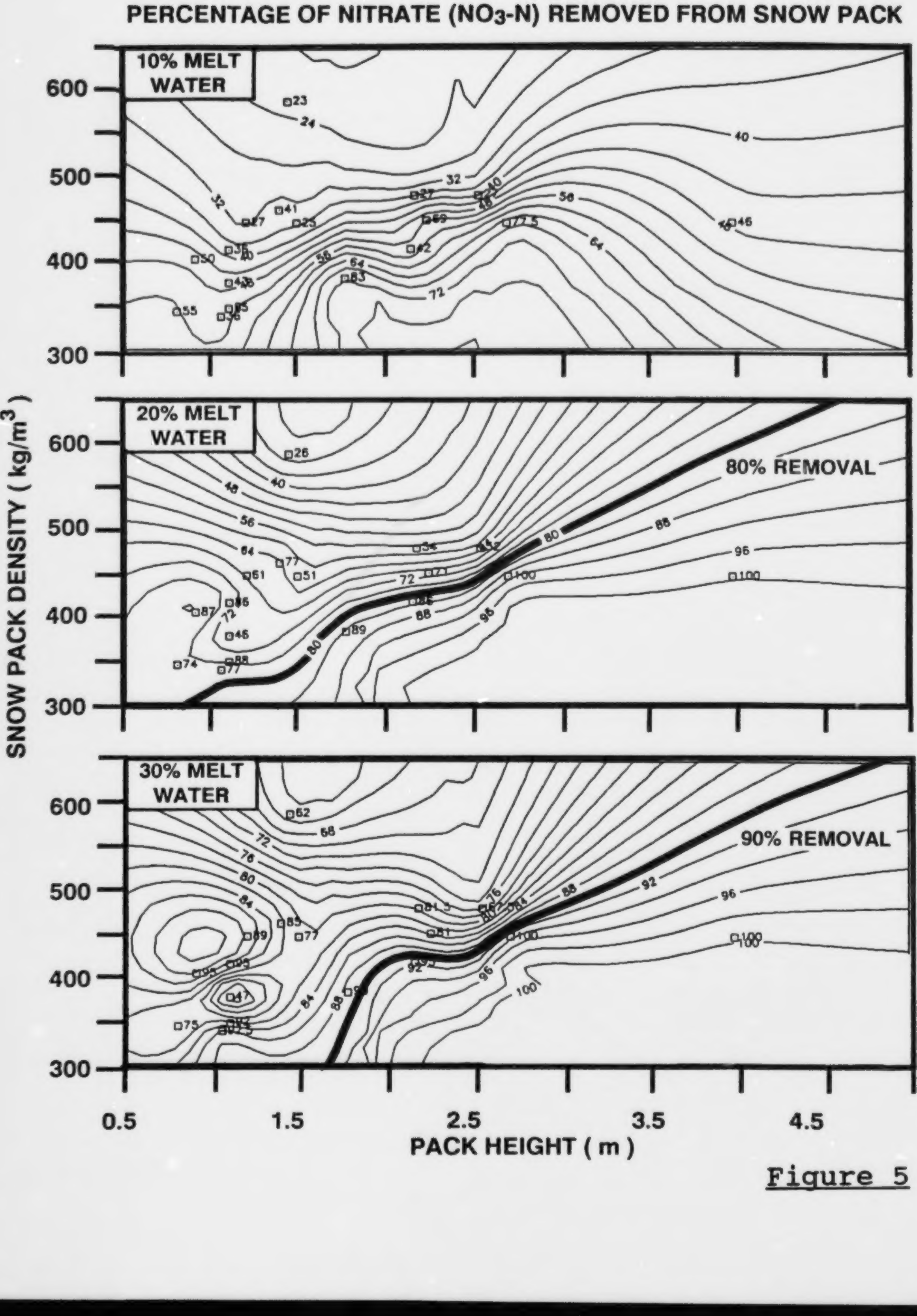
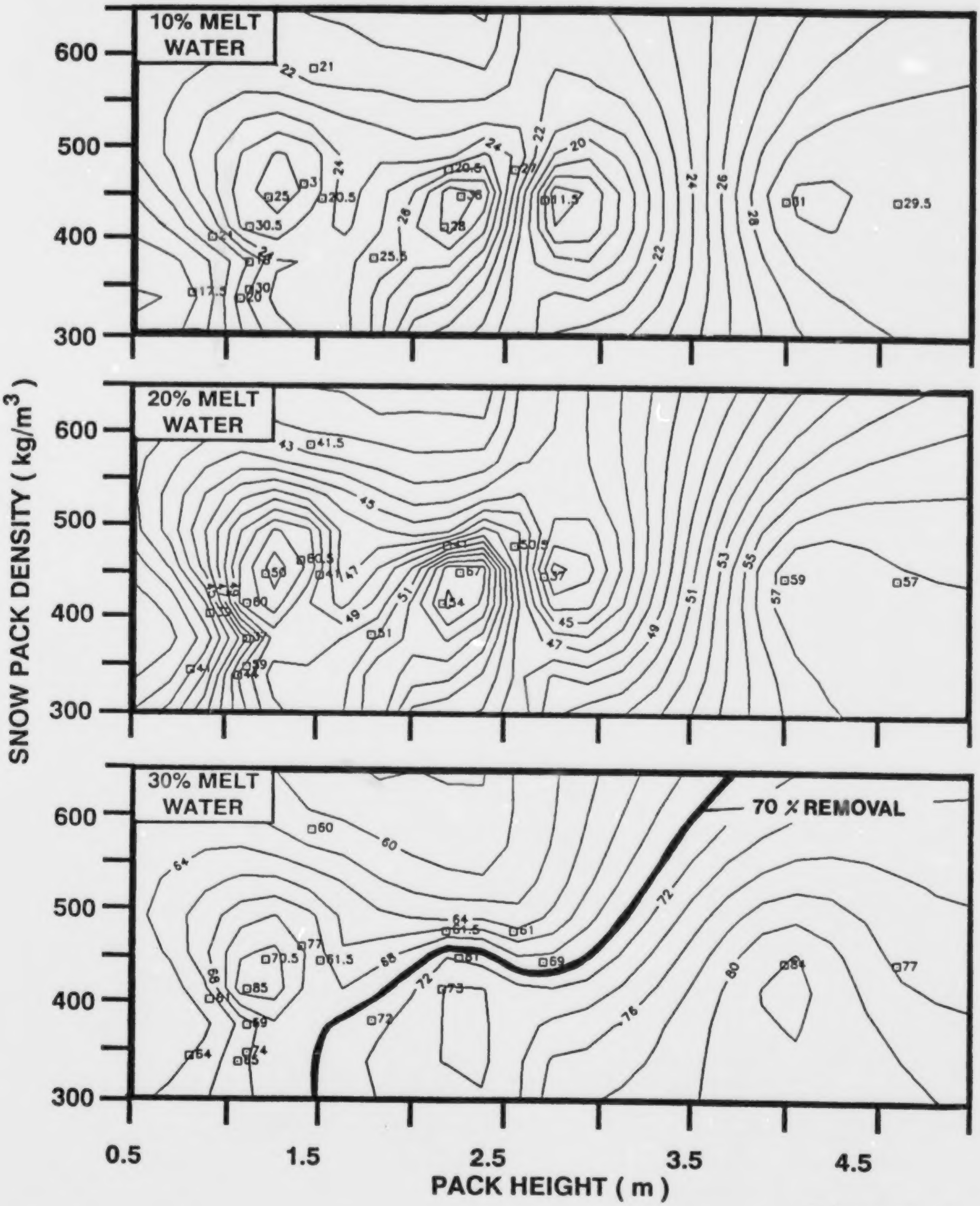

PERCENTAGE OF NITRATE (NO3-N) REMOVED FROM SNOW PACK
10% MELT WATER
20% MELT WATER
30% MELT WATER
80% REMOVAL
90% REMOVAL
SNOW PACK DENSITY (kg/m3)
PACK HEIGHT (m)
300
400
500
600
0.5
1.5
2.5
3.5
4.5

Figure 5



PERCENTAGE OF TOTAL PHOSPHORUS (TP) REMOVED FROM SNOW PACK

Figure 6

The phosphorus removal contour plots also show that the most efficient removal was achieved with pack heights above 2.0 metres and with snow densities below about 400 kg/m^3.

Multiple regression analysis confirms the observations made from the contour plots. For most of the impurities analyzed, both pack height and snow density have a statistically significant (95 percentile confidence level) effect on the degree of concentration. Generally, the concentration efficiencies improve with increased pack height and decreased snow density, with density being the more significant factor.

The TKN, nitrate, total phosphorus, silicon, sodium, potassium, and boron removal efficiencies are negatively correlated with density and positively correlated with pack height. The simple single factor regression analysis also indicates that snow density is a more significant variable than pack height for these impurities. For manganese and iron, the more significant design parameter was pack height rather than snow density.

APPLICATION OF FINDINGS

The research program demonstrated that the enhanced meltwater concentration phenomenon, previously reported in the literature, can have practical application for the disposal of effluent from municipal treatment systems. The technique has proven to be relatively robust in it's application, as the initial meltwater contains high nutrient concentrations for a wide range of snow densities and pack heights. However, for shallow pack depths (less than 2.0 metres) snow density can influence the process efficiencies, consequently it is recommended that snow densities less than 400 kg/m^3 be used for application purposes.

The following example demonstrates a simple design approach to the use of snowmaking techniques for effluent disposal:

1) $\mathbf{Q_a}$ = Applied wastewater flow
= 5,000 m^3/day

2) $\mathbf{T_n}$ = No. of non-irrigating days per year
= 180 days

3) $\mathbf{T_s}$ = No. of snowmaking days per year
= 60 days

4) $\mathbf{D_s}$ = Target water fraction of snow (density)
= 0.40 (i.e. 400 kg/m^3 or 40% water)

5) $\mathbf{A_s}$ = Soil absorptive capacity during snowmelt
= 0.3 m of water per month

6) $\mathbf{M_f}$ = Meltwater fraction to achieve desired removal
= 0.25 (25%)

7) $\mathbf{C_s}$ = Snowgun capacity
= 0.8 m^3/min

8) $\mathbf{E_a}$ = Application efficiency (Fraction of time of snowgun operation during cold weather)
= 0.75

9) $\mathbf{Q_s}$ = Total daily volume disposed through snowmaking

$$= \frac{Q_a \times T_n}{T_s} = \frac{5{,}000 \text{ m}^3\text{/day} \times 180 \text{ days}}{60 \text{ days}} = 15{,}000 \text{ m}^3\text{/day}$$

10) $\mathbf{V_s}$ = Total daily volume of snow produced

$$= \frac{Q_s}{D_s} = \frac{15{,}000 \text{ m}^3\text{/day}}{0.40} = 37{,}500 \text{ m}^3\text{/day}$$

11) $\mathbf{H_s}$ = Height of snowpack

$$= \frac{A_s}{M_f \times D_s} = \frac{0.30 \text{ m}}{0.25 \times 0.40} = 3.0 \text{ metres}$$

12) $\mathbf{L_a}$ = Land area required for snow effluent disposal

$$= \frac{V_s \times T_s}{H_s} = \frac{37{,}500 \text{ m}^3\text{/day} \times 60 \text{ days}}{3.0 \text{ m}} = 750{,}000 \text{ m}^2$$

= 75 ha

13) $\mathbf{G}$ = Number of snowguns required

$$= \frac{Q_s}{C_s \times E_a} = \frac{15{,}000 \text{ m}^3\text{/day}}{0.80 \text{ m}^3\text{/min} \times 1440 \text{ min/day} \times 0.75}$$

= 18 snowguns

The key design assumptions in the preceding example are:

1) the number of days in which snow can be made (T_s);
2) the operating time during those days ($E_a \times T_s$);
3) soil absorptive capacity during the melt (A_s).

These parameters affect the required land area for effluent snow disposal and the number of snowguns assuming that land for effluent irrigation is available for snow applications, the number of snow guns and the energy costs for pumping are the primary cost factors.

The snowgun used in the Kamloops research program was of a sophisticated design for use on ski-hills. The gun required liquid pressures in excess of 150 psi, and cost of approximately $20,000. Lower pressure snowmaking technologies are available which utilize compressed air to atomize and distribute the discharged liquid. A simple snowgun device could be developed specifically for effluent snowmaking. The gun should be designed for maximum throughput, minimum energy consumption and with very little emphasis on operational flexibility and snow quality. The development and use of such low pressure snow making equipment could reduce the capital cost considerably.

Operating costs are comparable to that for effluent spray irrigation, with energy requirements similar to effluent irrigation and nominal manpower requirements to adjust and or move the snowguns.

A conceptual effluent snow disposal facility might consist of irrigation towers, with snow guns mounted at the top. These towers could be utilized for both summer irrigation and winter snow production by modifying the discharge nozzle and providing a compressed air supply to the towers.

CONCLUSIONS

1) Snowmaking provides a viable means of effluent disposal for communities having effluent discharge problems during the winter months.

2) The initial meltwater from a snowpack contains a high percentage of the nutrients initially present in the snow. Metals are also concentrated in the early portion of the melt, but the degree of concentration varies between metals.

3) In general, the impurity removal efficiencies improve with decreasing snow density and increasing pack heights.

4) For practical applications it is recommended that snow densities be kept below approximately 400 kg/m^3 and pack heights above 2.0 metres.

5) Operating costs and land requirements for effluent snow making are similar to those for effluent spray irrigation.

6) Capital costs for snow applications may be reduced through the development of combined spray irrigation and snow making equipment systems.

7) Laboratory or pilot-scale studies should be carried out to test the concentration efficiency of the impurity of concern, prior to full-scale application.

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INNOVATIVE AND ALTERNATIVE WASTEWATER TREATMENT TECHNOLOGY EXPERIENCE IN THE UNITED STATES

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INTRODUCTION

Since 1972 the United States Environmental Protection Agency (EPA) has administered a multi-billion dollar grant program designed to assist local authorities in constructing the municipal wastewater treatment and conveyance facilities necessary to meet the nation ambitious water quality objectives. During this effort it became clear that special incentives would be necessary to promote technological innovation and the application of non-conventional treatment technologies that provide for more efficient use of resources. The Innovative and Alternative (I/A) Wastewater Treatment Technology Program was established in 1977 to provide these incentives and has resulted in the widespread utilization of innovative and alternative technologies for municipal wastewater treatment. This paper reviews the I/A technology program and the US experience with I/A technology.

Legislative History

The Federal Water Pollution Control Act Amendments of 1972 (PL92-500) commonly know as the Clean Water Act reflected the commitment of the United States Congress to restore and maintain the quality of the Nation's surface waters. The Act authorized EPA to make grants to local authorities for the planning, design and construction of publicly owned treatment works (POTW's) necessary to control municipal wastewater pollution. EPA's construction grants typically covered 75% of eligible project costs, which include most costs generally associated with POTW construction.

The 1972 Clean Water Act clearly established the intent of Congress to promote the construction of wastewater treatment facilities which recycle wastewater pollutants, reclaim wastewater, provide for the confined and contained disposal of pollutants not recycled and provide for the ultimate disposal of sludge in an environmentally sound manner. However the law did not provide for any type of financial incentive to promote construction of these types of facilities in lieu of conventional facilities. Hence the early years of the construction grants program were marked by construction of large regional treatment facilities employing conventional technology.

In 1977, Congress amended the Clean Water Act. Recognizing the need for bold action to promote the development and application of alternatives to conventional wastewater treatment Congress included provisions in the 1977 amendments establishing the Innovative and Alternative Technology Program as a 3 year experiment. Under this program communities utilizing I/A technology were eligible for grants covering 85% (rather than 75% as for conventional technology) of eligible project costs. In addition, to protect communities from the risk of employing nonconventional technology, the I/A program offered the possibility of grants for up to 100% of the cost of modification/replacement of I/A projects not performing to design expectations.

The I/A program's success led Congress, in the 1981 Clean Water Act Amendments, to continue incentives for I/A technology as a permanent feature of the construction grants program. The 1981 amendments made I/A technology even more attractive by reducing EPA construction grants funding for conventional technologies to 55% of eligible project cost while adjusting that for I/A technologies to 75%.

Under the 1987 Water Quality Act the I/A program will be phased out along with the construction grants program which will be replaced by a revolving loan program to be administered by the States.

I/A Program Objectives

The overall objective of the I/A program is to encourage the planning, design, and construction of cost-effective municipal treatment works that maximize the recycling and reclamation of water nutrients, and energy while minimizing adverse environmental and public health impacts. In addition to offering financial incentive to promote development and use of I/A technology the program has also pushed technology development and application through a vigorous research and development effort and a wide range of technology transfer activities.

I/A versus Conventional Technologies

Innovative technologies are wastewater treatment processes or components which are not fully proven in the circumstances of their intended use but, based upon documented research and demonstration projects, appear to offer the promise of benefits which outweigh potential risks of failure. Projects are designated as innovative on a case-by-case basis if they are significantly different from proven conventional or alternative technologies and if they offer the potential to significantly advance the state-of-the-art in terms of life cycle costs, environmental benefits, or more efficient use of energy and resources. An innovative technology includes an element of risk; however, the corresponding potential benefit outweighs the risk.

Alternative technologies are fully proven wastewater treatment systems that reclaim or reuse wastewater, productively recycle wastewater components, recover energy, or eliminate pollutant discharge. Specific alternative technologies include land treatment of wastewater and sludge, on-site treatment, alternative conveyance systems, composting, and methane recovery and use. Alternative technologies typically provide a cost savings compared to conventional treatment, because of lower operation and maintenance costs or cost recovery through productive use of wastes.

As a basis of comparison conventional technology is defined as a proven wastewater treatment process or established technique involving the treatment of wastewater at a centralized treatment plant followed by direct point source discharge to surface waters.

PROGRAM STATUS

From the I/A program's inception through 1987 EPA invested nearly $4.4 billion in I/A facilities for municipal wastewater treatment. State and local governments have invested an additional $1.0 Billion. During the same period EPA invested a total of $23 billion in all types of POTW's. Thus nearly 20% of EPA's construction grant funds have been invested in I/A facilities since the I/A program began operating in 1979. Some 2,700 I/A technology projects have been funded: 2,100 alternative projects in which the Federal investment totals $3.3 billion and 600 innovative projects in which the Federal investment totals $1.1 billion. Of the 2,700 I/A technology projects funded, nearly half were in operation at the end of 1987.

I/A TECHNOLOGY EXPERIENCE

Alternative Technology

Nearly 14 percent of the total dollars invested by EPA in all types of municipal wastewater systems from 1979 through 1987 have been for alternative projects. Just over 30 percent of all construction grant projects funded during this period have been alternative. Alternative technology has been a particular boon to small communities. Of the 2100 alternative projects funded through 1987, nearly half were sized for communities of less than 3,000 in population; nearly two-thirds were sized for communities of less than 10,000.

Table 1 provides a summary of alternative technology utilization in the United States by State and technology. Figure 1 presents a graphical depiction of alternative technology utilization. From Figure 1 it may be seen that 29 percent of the alternative projects funded involve some aspect of land treatment of wastewaters (aquacultive/wetlands, overland flow, rapid infiltration, slow rate irrigation, preapplication treatment/storage, or other land treatment system); 24 percent involve sludge treatment (land spreading, preapplication treatment, composting, or other sludge treatment/disposal method); 20 percent involve collection systems (effluent pump, grinder pump, small diameter gravity sewers, or vacuum sewers); 11 percent involve on-site treatment (single family septic tanks/soil absorption, mounds, evapotranspiration beds, aerobic units, sand filters, multiple family septic tank/soil absorption, septage treatment and disposal, or other on-site systems); 8 percent involve energy recovery from sludge (anaerobic digestion with 90% methane recovery or self-sustaining incineration); and 7.5% involve either aquifer recharge, direct nonpotable reuse, or total containment ponds.

Certain alternative technologies have proven exceptionally popular. Land spreading of POTW sludge accounts for 366 or nearly 1/5 of all alternative projects. Slow rate irrigation accounts for 294 or 14 percent. Other very popular specific technologies are small diameter gravity sewers and grinder pump pressure sewers with 157 and 152 projects respectively or each representing about 8 percent of all projects funded; anaerobic digestion with 90 percent methane recovery for 152 projects or about 8 percent of the total; and total containment ponds account for 132 projects or just over 6 percent of all alternative projects funded.

One of the most potentially significant impacts of the I/A program involves what it has done to advance the level of professional and public acceptance of alternative technologies. Prior to the I/A program about 19 new land treatment projects per year were initiated in the United States. Under the I/A program this number has jumped to 57 projects per year. The number of sludge composting projects increased from about 15 in 1980 to nearly 200 today. The number of land spreading of sludge projects increased from 1700 in 1980 to 2700 in 1986. Alternative conveyance systems (pressure sewers, small diameter gravity sewers, and vacuum sewers) were virtually unknown in the United States before the I/A program. However, by the end of 1987, a total of about 400 such systems were in some stage of development. While not all of these projects were funded under the I/A program, the program's active research and development, and technology transfer efforts greatly facilitated many of those that the program could not fund.

The United States experience with alternative technology has been extremely positive. With very few exceptions alternative technology projects have fully performed to design expectations and have provided cost-effective, environmentally sound wastewater treatment. Alternative technologies such as land treatment of wastewater, land spreading of sludge, alternative conveyance systems, sludge composting, and on-site systems have moved from relative obscurity to wide spread acceptance and application. Nearly 1100 alternative technology projects are currently in operation. Notable performance problems have been experienced at only 29 facilities (3 percent of those operating). These performance problems are further discussed following the next section of this paper.

Innovative Technology

Nearly 5 percent of the total dollars invested by EPA from 1979 through 1987 in all types of municipal wastewater systems have been for innovative projects. Innovative projects have represented nearly 9 percent of the total number of projects funded during this period. Not surprisingly, innovative technologies tend to be more popular among communities larger than those favoring alternative systems. About half of all innovative projects are designed for communities of more than 10,000 people; with only 20 percent designed for communities of less than 3,000.

Table 2 provides a summary of innovative technology utilization in the United States by State and Technology. Figure 2 presents a graphical depiction of innovative technology utilization. Approximately 120 specific technologies have been designated as innovative. As may be seen in Table 2 and Figure 2 these technologies are distributed among 11 major groupings. The major groupings are aeration/mixing (83 projects or 14% of the total); lagoons (65 projects or 11% of the total); clarifiers (63 projects or 11% of the total); sludge technologies (27 drying beds/land application, 5 incinerator, 8 composting, and 23 digestion for a total of 63 or 11% of all innovative projects); and disinfection (50 projects or 8% of the total).

The true long-term impact of the innovative part of the I/A program relates to the technologies which it has moved from research and development toward relatively widespread, full-scale application. Two typical examples are ultraviolet disinfection and sequencing batch reactors.

Ultraviolet disinfection, at the beginning of the I/A program, was generally considered a promising experimental technology. Over the course of the program, nearly 50 ultraviolet disinfection projects were funded. The successes and problems experienced with these projects served to benefit succeeding projects. Today ultraviolet disinfection is a state-of-the-art municipal wastewater disinfection technology.

Sequencing batch reactors are another example of the I/A program's success. The basic concept of the sequencing batch reactor had been around for most of this century. However, the process had never gained popularity because it was difficult to control. However, under the I/A program, the ability of modern electronic and mechanical hardware to greatly simplify process control was demonstrated on a wide scale. About 20 sequencing batch reactor projects have been funded under the I/A program. The sequencing batch reactor is now generally considered a viable wastewater treatment option for small to mid-size communities.

The United States experience with innovative technologies has been generally positive. Innovation inherently involves risk and not all innovative projects have performed to design expectations. Of the 300 operating innovative projects 41 (15 percent) have experienced notable performance problems. These are further discussed in the next section.

Problem Technologies

As mentioned in the preceding sections approximately 3 percent of operating alternative and 15 percent of operating innovative projects have experienced notable performance problems. In all some 31 separate technologies have experienced performance problems as part of one or more projects. Seven technologies have experienced performance problems as part of three or more projects and these will be the focus of this discussion. Table 3 lists those I/A technologies for which 3 or more projects have reported notable performance problems.

Community mound systems have experienced the largest number of problems. Generally these problems have resulted from attempts to design community mounds based upon simple scale-ups of individual single family mounds. Experience has shown that successful design of multifamily mounds involves additional hydraulic considerations not significantly considered in single family mound design.

To date 7 draft tube aeration systems have experienced significant performance problems. These problems have generally been associated with actual oxygen transfer efficiencies being substantially less than design expectations. This has led to higher than anticipated operating costs.

Six rapid infiltration projects have experienced performance problem. These problems are associated with taking a proven technology and increasing the extent and circumstances of its application. The designs of the problem systems did not adequately account for the volume of wastewater that can be applied and infiltrated within the time allowed. The problems can be grouped into four major categories related to: soil conditions, groundwater conditions, design assumptions, and construction control. Careful analysis of each of these areas can avoid problems with future systems.

Ash accumulation has plagued 5 projects in which wet air regeneration of powdered activated carbon was utilized. The original design concept for ash removal did not perform as expected and ash accumulated in these systems resulting in ash related effluent quality problems. Solutions to this problem have involved additional complex processes as well as abandonment of WAR facilities.

Inadequate disinfection was experienced by 4 ultraviolet disinfection systems. These were among the early systems installed. Significant improvements have been made in ultraviolet system design since that time as our understanding of this technology has became more sophisticated.

TABLE 3
Problem I/A Technologies

Technology	Number of problem projects	Nature of problems
Community Mound Systems	8	Insufficient sizing for hydraulic load or inadequate site investigations.
Draft tube aerators	7	Oxygen transfer efficiencies less than design resulting in higher operating costs.
Rapid infiltration	6	Inadequate site investigation or construction control.
Ultraviolet Disinfection	4	Inadequate disinfection.
Wet air regeneration as an adjunct to Powdered Activated Carbon Treatment	5	Ash buildup in systems leading to effluent problems.
Composting	3	Inadequate ventilation in enclosed buildings, odor control, or need for additional bulking agent.
Intrachannel Clarifiers	3	Inadequate mixing in oxidation ditch or clarifier control problems.

Three composting projects experienced performance problems. Two of these are associated with inadequate ventilation and odor control. It is now recognized that if a composting operation is to be conducted in an enclosed building careful consideration must be given to ventilation and moisture control.

Three intrachannel clarifier projects have experienced performance problems. Generally these problems have been associated with inadequate mixing in the oxidation ditch. EPA is currently preparing a technology evaluation of intrachannel clarification.

TECHNOLOGY TRANSFER

From the outset technology transfer has been an integral aspect of the I/A program. Good communication have been largely responsible for the progress to date in acceptance and use of I/A technologies.

The cornerstone of the I/A program's successful technology transfer efforts has been a network of EPA Regional and State I/A coordinators. These individuals were responsible for promoting the program and serving as information sources. In turn the State and Regional coordinators were served by a National I/A coordinator at EPA Headquarters and a staff of I/A technology experts at EPA's research laboratory in Cincinnati, Ohio. This network of individuals was able to effectively collect and disseminate information about I/A technologies.

The I/A program has produced a legacy of outstanding technical literature. Of particular note are several dozen major I/A technology research reports and Process Design Manuals as well as nearly 50 brochures and booklets describing various I/A technologies.

The National Small Flows Clearinghouse at West Virginia University has also played a significant role in I/A technology transfer. The clearinghouse maintains the I/A database which contains key information about grant funded I/A projects. They also maintain bibliographies of information on some I/A technologies.

EPA has also conducted or participated in numerous international, national, and regional meetings and seminars related to wastewater technology.

SUMMARY AND CONCLUSIONS

The I/A program has been tremendously successful at promoting the development and application of more cost effective, environmentally sound wastewater treatment technology. Through financial incentives, an active research and development effort and an aggressive technology transfer program the I/A program has significantly advanced the professional and public acceptance of I/A technologies.

The I/A program, in addition to directly helping nearly 3000 communities effectively solve their wastewater treatment problems, leaves a legacy of technical understanding which will continue to benefit many additional communities in the United State and around the world, for years to come.

Figure 1
Alternative Technologies Funded
(Percent of Total Number of Facilities)

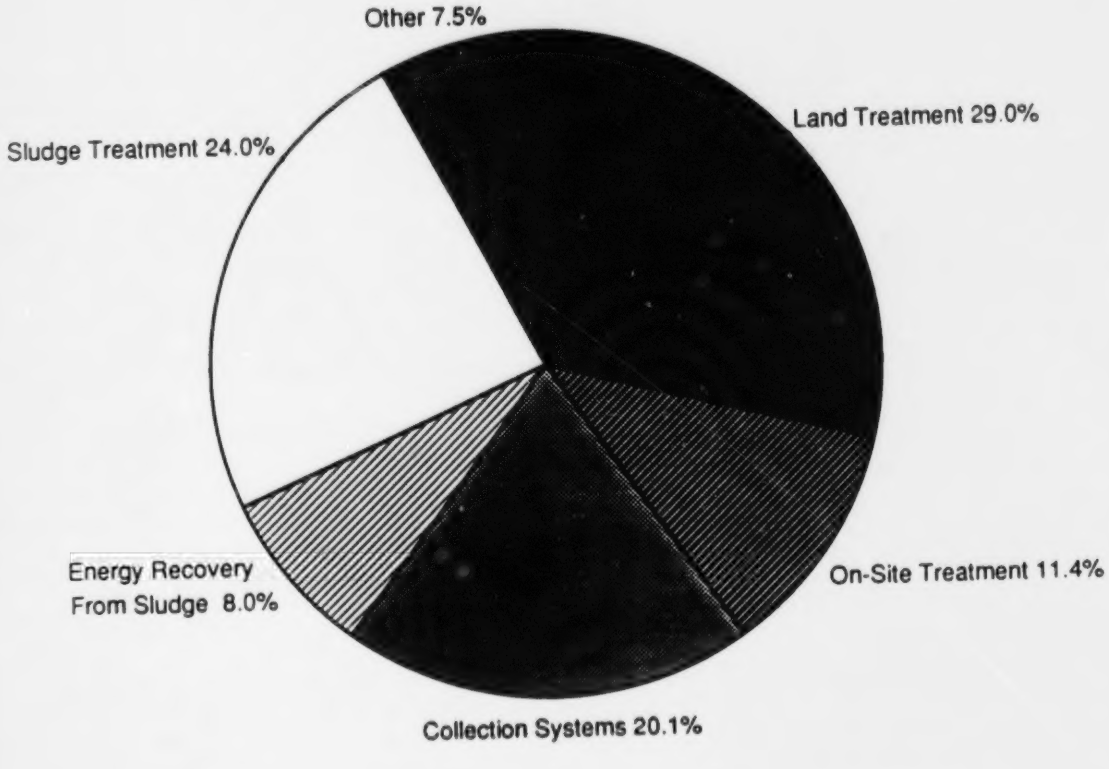

Table 1

SUMMARY OF ALTERNATIVE TECHNOLOGIES

	ON-SITE TREATMENT								LAND TREATMENT					
STATE	Septic Tank/ Soil Absorption (Single Family)	Mounds	Evapotranspiration Beds	Aerobic Units	Sand Filters	Septic Tank/ Soil Absorption (Multiple Families)	Septage Treatment and Disposal	Other On-Site Treatment	Aquaculture/ Wetlands Marsh	Overland Flow	Rapid Infiltration	Slow Rate Irrigation	Preapplication Treatment or Storage	Other Land Treatment
Alabama												2		
Alaska						2							1	
Arizona									3		1	12	1	
Arkansas	1						1					4	2	1
California		1				7	3		3	2	14	20	24	
Colorado												2	1	1
Connecticut					1	2	7				1			
Delaware						2					1			
Florida										1	2	20		
Georgia										2		20		
Guam											2			
Hawaii												2		
Idaho					2			1	1		4	9	10	1
Illinois	5	1			13					3	1	3		3
Indiana		2				2								1
Iowa					1				3			2	3	
Kansas									1		1	16	9	
Kentucky	1				2	2			2	1		2		
Louisiana				1						6		2	2	
Maine	3				5	5						1		
Maryland	3	2			2		3	1	2	2	1	5	4	2
Massachusetts					1		19				2	1		
Michigan	2					6			1	1	3	13		
Minnesota	8	8			2	5		1			1	14	11	
Mississippi										11		2		
Missouri		1			1					9		15		
Montana											3	8		
N. Mariana Islands														
Nebraska									1		2	5		
Nevada											5	6	5	
New Hampshire					4	3	7			2				
New Jersey						1	11						1	
New Mexico												6	5	
New York	1	2			12	2	4	1		2	3			
North Carolina												21		
North Dakota		4								1		6		
Ohio	2			1			3					1		1
Oklahoma								1				31	16	
Oregon			2		3				2		1	6	9	1
Pennsylvania	3	1		1	1	2			2		1	5	2	1
Puerto Rico														
Rhode Island							2							
South Carolina											1	9		
South Dakota									5		8	1	3	
Tennessee	2									2		1	4	
Texas	2									1	1	11	10	1
Trust Territories						6		1						
Utah												3	2	
Vermont					2	1						1		
Virgin Islands														
Virginia							3			3	1	1		
Washington		1				2			1			3	4	2
Washington, DC														
West Virginia	1							1						
Wisconsin		2									8		5	9
Wyoming											3	2		
Totals	34	25	2	3	52	50	63	7	27	49	71	294	134	24

Table 1 (cont'd)
SUMMARY OF ALTERNATIVE TECHNOLOGIES

STATE	COLLECTION SYSTEMS				ENERGY RECOVERY/SLUDGE		SLUDGE TREATMENT				OTHER			Totals
	Pressure Sewers/ Effluent Pump	Pressure Sewers/ Grinder Pump	Small Diameter Gravity Sewers	Vacuum Sewers	90% Methane Recovery/ Anaerobic Recovery	Self-Sustaining Incineration	Land Spreading of POTW Sludge	Preapplication Treatment	Composting	Other Sludge Treatment or Disposal	Aquifer Recharge	Direct Reuse	Total Containment Ponds	
Alabama	1	2	3		3		2		2					15
Alaska	1		1		1	1	1		1	1				10
Arizona					3		1			1		1	1	24
Arkansas		10	2		1		2							24
California	8		4	1	5	1	2	2	1	2		2	1	103
Colorado		1			1		2				1			9
Connecticut					4	1			1					17
Delaware		2	1				2	1	2					11
Florida					3			1	2		1	3		33
Georgia	2		1		3	1	5							34
Guam														2
Hawaii													1	3
Idaho	2		2		3		6						1	42
Illinois	5	2	18		13		47	7		5		4		130
Indiana	2	4	13		5		19							48
Iowa	2	3	1		6		24	2						47
Kansas					5	2	20		1	2			27	84
Kentucky	2	4	5	2	2		13							38
Louisiana	1		1				1							14
Maine	1	1							6					22
Maryland	2	21	3	2	2		4		5					66
Massachusetts			1		2	2	1		4					33
Michigan	1	2	1		4		10							44
Minnesota	8	6	7		9		24	1	2					107
Mississippi	1	3	1				3			1				22
Missouri	6	•18	15		1		27			8				101
Montana					2		9			1			5	28
N. Mariana Islands			1											1
Nebraska					3		5	3	2	2			32	55
Nevada					3								4	23
New Hampshire		1							1					18
New Jersey		2		2			1	3	5	1				27
New Mexico					1					1				13
New York	3	16	16	2	16	1	2		2	1				86
North Carolina	1	2	1		6		4			1				36
North Dakota	3	2	14										17	47
Ohio	2	5	2		11		34	3	3					88
Oklahoma						1	5	2	1				29	86
Oregon	5		4		5		4	3	1	2			1	49
Pennsylvania	6	15	11		3		5		3	2				64
Puerto Rico									1					1
Rhode Island						1			1					4
South Carolina			2		1		5							18
South Dakota	1	1	2		4		11	1				1	7	45
Tennessee	5	6	10	2			5		1	1				39
Texas	3	4	1		6		20	8	1	1		4	1	75
Trust Territories			1											8
Utah					1		1		1					8
Vermont		3	1				12							20
Virgin Islands														0
Virginia	3	2	2		5	2	9	1	3	3		5		43
Washington	2	1	1		2		1	1					2	23
Washington, DC									1					1
West Virginia	6	10	5	10	2									35
Wisconsin	1	3	3		4		15			1				51
Wyoming					1		2						3	11
TOTALS	86	152	157	21	152	13	366	39	54	37	2	20	132	2066

Figure 2
Innovative Technologies Funded
(Percent of Total Number of Facilities)

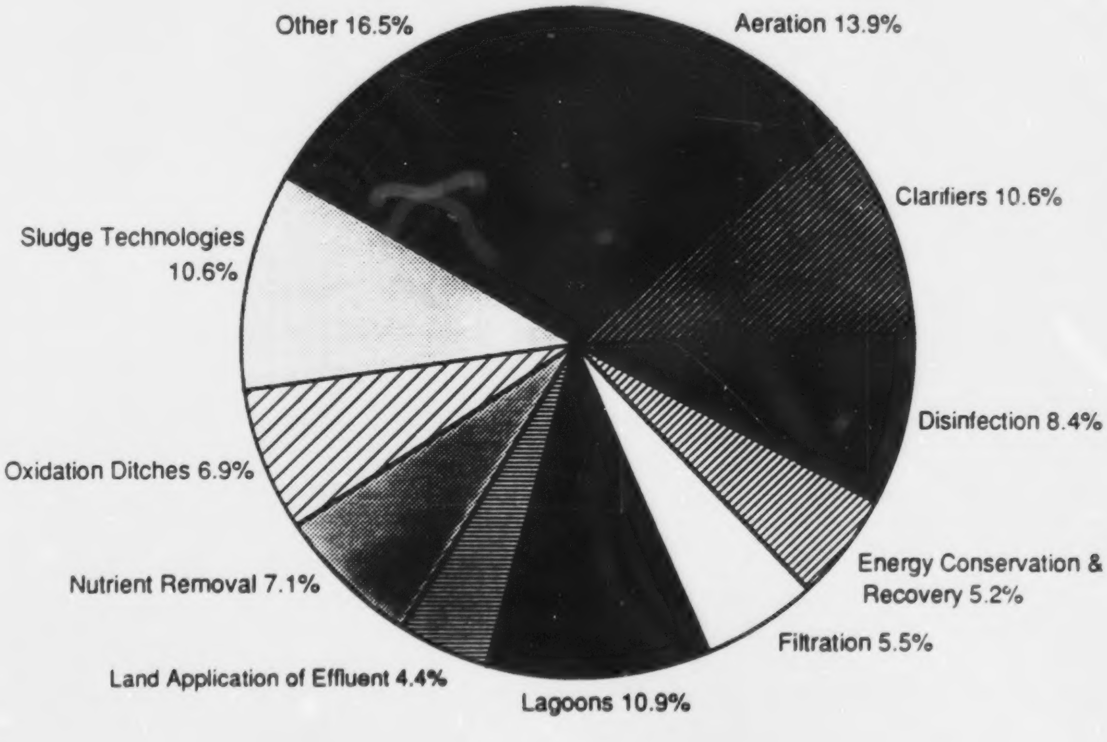

Table 2

SUMMARY OF INNOVATIVE TECHNOLOGIES (1)

	WASTEWATER TREATMENT										SLUDGE TREATMENT				Misc. (2)	Totals
State	Aeration/ Mixing	Clarifiers	Disinfection	Energy Conservation and Recovery	Filtration	Lagoons	Land Treatment	Nitrification	Nutrient Removal	Oxidation Ditches	Drying Beds/ Land Application	Incineration	Composting	Digestion		
Alabama	11	4	1		1	5				5	1		1		2	31
Alaska						1					1	1		2	1	6
Arizona		1	2	1		1			1		1				1	8
Arkansas		1	1		2	2	3		2						2	13
California		2	1	3	3	1		1	1		4			1	2	19
Colorado	2		1		1			1	1		1					7
Connecticut	1		2													3
Delaware	1	1								1						3
Florida	2					5			1				1		3	12
Georgia	1	1		2			1									5
Guam																0
Hawaii															1	1
Idaho	1	1			1	1				1				1	2	8
Illinois	1	3	1		3	12	2		1	1	1				1	26
Indiana	2	1				10		1			1				3	18
Iowa	2	1	1	1					3	1				3		12
Kansas	1	1	3			2				1	2				1	11
Kentucky	1	7			1	2	2				1		1		2	17
Louisiana		4	1			3	1			4						13
Maine	2	1	3	1	1	3	1								3	15
Maryland	1	1	4						4					2		12
Massachusetts	1		2	1				2	1						3	10
Michigan	1	1			1				1	1					3	8
Minnesota	1	1	5	7	1	1	1	1	3	1	1			2	1	26
Mississippi		1				8	1		1						1	12
Missouri	2	4	2	1	2							1			4	16
Montana		1	1				1		2					1	2	8
N. Mariana Islands																0
Nebraska									1		1				2	4
Nevada				1					2							3
New Hampshire			1													1
New Jersey							1			1	3				8	13
New Mexico	1				1										1	3
New York	6	2	4	1	1			1	2	5		1	1		4	28
North Carolina	8				1					2	1	1		1	2	16
North Dakota							1									1
Ohio	3	4	3	3	3					1	1		1		11	30
Oklahoma	1	2	2	1	1		1			2	2			4	2	18
Oregon	1	1		1			1				1		1		1	7
Pennsylvania	3	1			2		1		4	2		1		2	3	19
Puerto Rico																0
Rhode Island	2			1												3
South Carolina	1	1		1	1	1	1			1	1		2	2	1	13
South Dakota	3					1	1								1	6
Tennessee	11	3			1	2	1		1	3				1	2	25
Texas	1	1				2	3			1	2				8	18
Trust Territories																0
Utah		1					1									2
Vermont			1	1											1	3
Virgin Islands																0
Virginia	2	3	2		3	1	1	1	1	6	1				2	23
Washington			1	1											1	3
Washington, DC															4	4
West Virginia	3	4							1	1				1	1	11
Wisconsin	2	2	1	3	1										3	12
Wyoming	1		4		1	1									2	9
Total	83	63	60	31	33	65	26	8	34	41	27	5	8	23	98	595

(1) Includes numerous Specific Technologies. See Appendix A.
(2) Miscellaneous category includes both sludge and wastewater management technology which could not be separated using available data.

Feasibility of Biological Phosphorus Removal at Lakeview WPCP

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A full-scale demonstration study on biological phosphorus (bio-P) removal was conducted at the Lakeview Water Pollution Plant (WPCP) in Mississauga, Ontario. The objectives of the study were to examine the feasibility of bio-P removal at the plant, assess the potential benefits of bio-P removal, and estimate a payback period for a bio-P removal retrofit of the entire plant. The results of the study indicated that a bio-P removal retrofit was possible at the plant. The potential benefits of a bio-P retrofit include at 16% reduction in aeration power requirements, a 70% reduction in chemical use, and a 17% reduction in sludge production. The estimated payback period for a bio-P removal retrofit on the entire plant is 2.5 years.

Introduction

Phosphorus removal is generally achieved in municipal wastewater treatment plants through physicochemical precipitation with metal salts, such as alum, ferric chloride and lime. This method has proved reliable in reducing plant effluent total phosphorus concentrations to 1 mg/L and less. However, as effluent requirements become more stringent, the costs for chemical precipitants increase dramatically.

The phenomenon of biological phosphorus (bio-P) removal by activated sludge in excess of that required for normal cell metabolism was first reported in 1955[1]. The advantages of utilizing biological as opposed to physicochemical methods of phosphorus removal in the treatment of wastewater were recognized immediately. They include reduced chemical costs, reduced aeration costs, and improved sludge handling characteristics.

Subsequent research has led to an increased understanding of the mechanisms responsible for the bio-P removal process[2-7]. The process consists of an anaerobic stage followed by an aerobic stage. In the anaerobic stage, polyphosphate (PolyP) organisms assimilate organic matter and convert it to intermediate storage products, including poly-B-hydroxybutyrate (PHB), while concurrently releasing phosphates[8,9]. The PolyP organisms have the ability to store polyphospate in quantities larger than normally required for cellular functions[10]. The release and uptake of phosphates was initially attributed to the Acinetobacter strain of bacteria, but more recent findings have shown that other organisms, such as Aeromonas punctata, also play a vital role in the process[11].

During the aerobic stage, the stored PHB acts as an energy source for the synthesis of new cells. The previously released phosphates are accummulated in the new cells resulting in a sludge with a high phosphorus content[4]. The phosphorus is removed from the system by wasting a portion of the sludge, as in a normal waste activated sludge system.

Several full-scale studies have been conducted on existing municipal wastewater activated sludge treatment plants to determine the feasibility of retrofitting them for bio-P removal[12,13]. Each retrofit situation, however, is site-specific and a plant should be given individual consideration in terms of wastewater characteristics, operational difficulty, and cost effectiveness before a retrofit is implemented.

The Lakeview Water Pollution Control Plant (WPCP) in Mississauga, Ontario currently achieves phosphorus removal by precipitation with ferrous chloride. An assessment of the wastewater to the plant indicated that it met the requirements for a bio-P removal retrofit[7]. On this basis, the plant was chosen as the site for a full-scale demonstration study.

Objectives

The objectives of this study were as follows:

i) to examine the feasibility of bio-P removal at the Lakeview WPCP;

ii) to assess the potential benefits of bio-P removal; and

iii) to estimate a payback period for a bio-P removal retrofit of the entire plant.

Materials and Methods

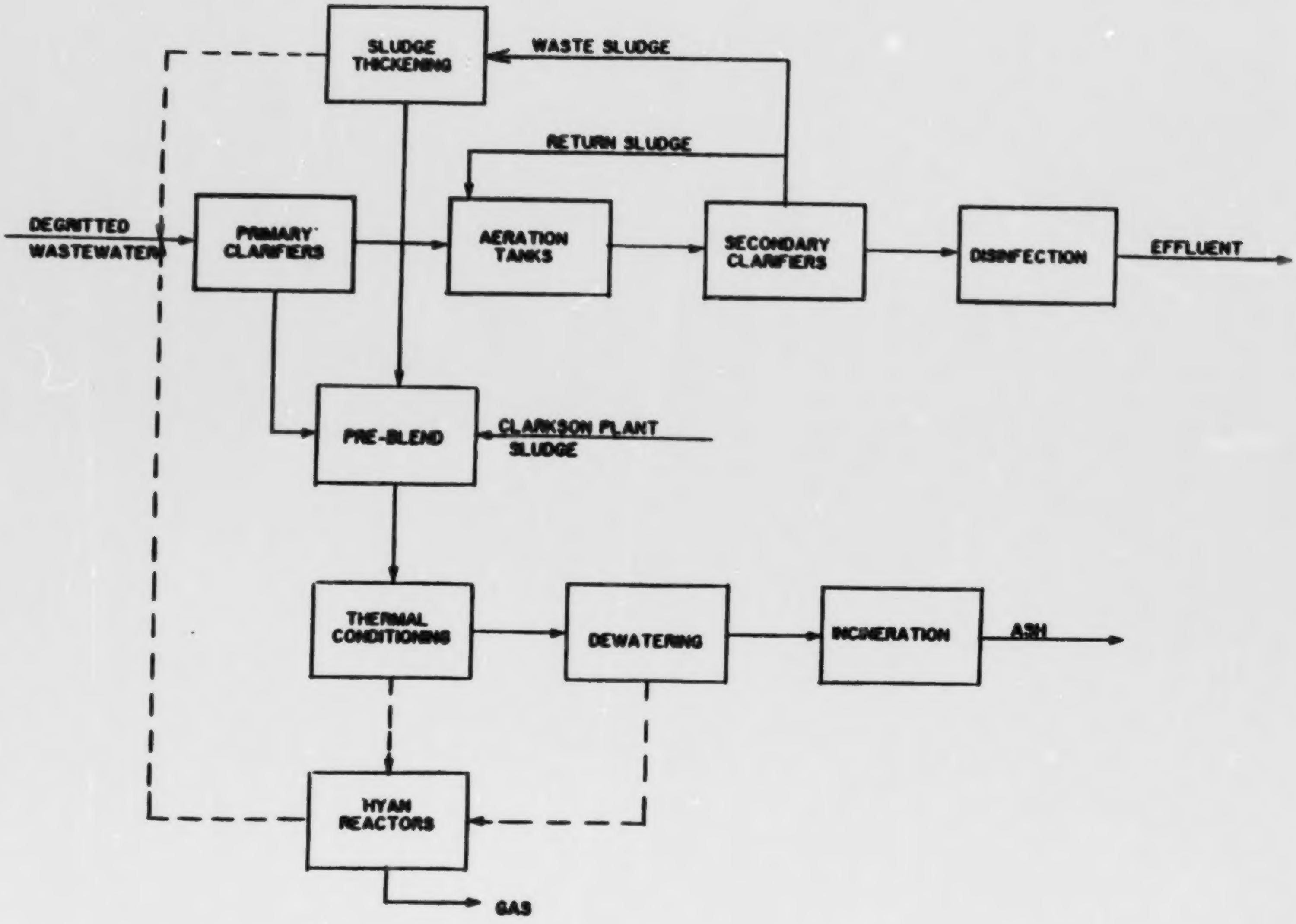
Lakeview WPCP

The Lakeview WPCP is located in the southeast section of the City of Mississauga, Ontario. The plant was constructed in 1961 with a design capacity of 22 500 m^3/d. An extension in 1967 doubled the capacity to 56 000 m^3/d. A second plant was added in 1972 which increased the capacity to 168 000 m^3/d. In 1975, a third plant was added which brought the total facility capacity up to its present value of 280 000 m^3/d.

Figure 1 is a process flow schematic of the Lakeview WPCP. The wastewater undergoes primary and secondary treatment including screening and grit removal, primary settling, aeration, final settling, chemical phosphorus removal and disinfection prior to discharge to Lake Ontario. The secondary sludge undergoes thickening by centrifuge after which it is combined with the primary sludge and sludge from the Clarkson WPCP. The sludge is subject to thermal conditioning and dewatering prior to incineration. Sidestreams from the thermal conditioning unit and the dewatering facility are anaerobically treated in Hyan reactors and recycled to the plant headworks along with centrate from the centrifuge sludge thickeners.

Phosphorus removal is currently achieved at the Lakeview WPCP by precipitation with ferrous chloride. A ferrous chloride solution is added to the activated sludge secondary treatment sections to meet the effluent requirement of 1 mg/L or less of total phosphorus.

The aeration tanks are spiral flow, three pass tanks with diffusers located along one side. During normal operation, the air supply to the tanks is capable of creating a spiral flow velocity of 1.2 to 1.8 m/s at the tank floor.


SLUDGE THICKENING
WASTE SLUDGE
RETURN SLUDGE
DEGRITTED WASTEWATER
PRIMARY CLARIFIERS
AERATION TANKS
SECONDARY CLARIFIERS
DISINFECTION
EFFLUENT
PRE-BLEND
CLARKSON PLANT SLUDGE
THERMAL CONDITIONING
DEWATERING
INCINERATION
ASH
HYAN REACTORS
GAS

PROCESS FLOW SCHEMATIC
LAKEVIEW WPCP

FIGURE 1

Modifications for Bio-P Removal

The bio-P study was conducted in Plant 1 and Extension 1. Plant 1 consists of two equally sized aeration tanks (Number 1 and 2) and four final clarifiers (Final 1 to 4). Extension 1 consists of two equally sized aeration tanks (Number 3 and 4) and two final clarifiers (Final 5 and 6). Both plants receive primary effluent from a common channel. Each plant has a separate return sludge facility. The basic design parameters of Plant 1 and Extension 1 are shown in Table 1.

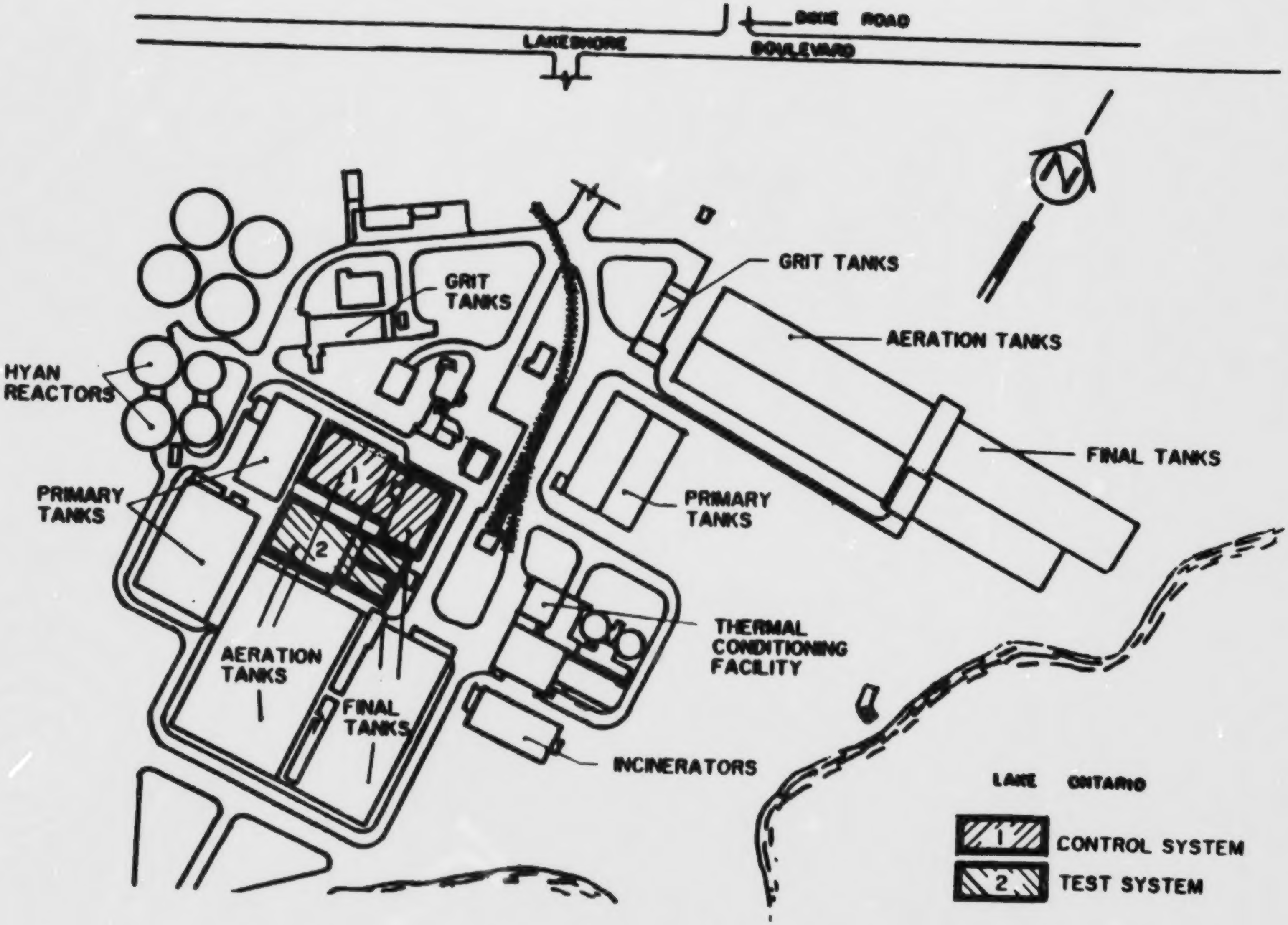
Plant 1 was operated normally as a control system and Extension 1 was used as a test system for the bio-P study. The capacity of each plant is 22 500 m^3/d, or about 10% of the total plant capacity. Had the bio-P system failed during the study, the impact on the overall Lakeview WPCP effluent would, therefore, have been limited.

Table 1: Design Parameters of Plant 1 and Extension 1

Component	Design Parameters	
	Plant 1 (Control)	Extension (Test)
Aeration Tanks		
Number of tanks	2	2
Total tank volume (m^3)	7,200	6,900
Final Clarifiers		
Number of tanks	4	2
Tank area (m^2)	1,126	966
Tank volume (m^3)	3,770	3,540
Weir length (m)	284	338

An anaerobic zone was introduced in the first pass of the aeration tanks of Extension 1 by shutting off the air supply. Mixing was maintained in the tanks through the installation of three 5.6 kW mixers. Due to the fact that there were only three mixers available, the two aeration tanks in the bio-P demonstration system were operated with different configurations. Aeration Tank 3 had two mixers in the first two-thirds of the first pass while Aeration Tank 4 had one mixer placed in the first one-third of the first pass. The aeration was reduced to provide a liquid velocity of approximately 0.3 m/s in the next one-third to prevent any solids from settling. The location of the mixers and the dissolved oxygen levels at various points along the aeration tanks used in the study can be seen in Figure 3.

Once the aeration tanks in Extension 1 were properly configured for bio-P removal, the addition of ferrous chloride was discontinued. Monitoring of the two plants began four days, or approximately one solids retention time after that time.


DIXIE ROAD
LAKESHORE
BOULEVARD
N
GRIT TANKS
AERATION TANKS
FINAL TANKS
GRIT TANKS
HYAN REACTORS
PRIMARY TANKS
PRIMARY TANKS
1
2
AERATION TANKS
FINAL TANKS
THERMAL CONDITIONING FACILITY
INCINERATORS
LAKE ONTARIO
1 CONTROL SYSTEM
2 TEST SYSTEM

LAKEVIEW WATER POLLUTION CONTROL PLANT FIGURE 2

Monitoring Program

The primary effluent and the effluent from Plant 1 and Extension 1 were monitored for the duration of the study. Operating parameters, such as detention time, F/M ratio, and SVI, were monitored in both plants as part of the regular plant operation. As the operating conditions of the two aeration tanks in Extension 1 were operated with different configurations, they were monitored individually. Samples taken were 24 hour composites, except as indicated. The parameters monitored, sampling locations, and weekly sampling frequency are listed in Table 2.

Table 2: Sampling and Analytical Program for Bio-P Study

	Sampling Points and Weekly Frequency						
		Aeration Tanks			Final Clarifiers		
	Primary Effluent	1 - 2	3	4	1 - 4	5	6
Flow	4				4	4	4
BOD (total)	4				4	4	4
BOD (soluble)	4				4	4	4
SS	4				4	4	4
P (total)	4				4	4	4
P (soluble)	4				4	4	4
Recyle Flow		4	4	4			
MLSS *		4	4	4			
MLVSS *		1	1	1			
Waste Sludge Flow		4	4	4			
Waste Sludge Solids*		4	4	4			
SVI		4	4	4			

* Grab samples

All analyses were performed according to Standard Methods[14].

Results and Discussion

As previously mentioned, Plant 1 was used as a control system and Extension 1 was operated as test system for the duration of the study. To ensure that both plants were operating under comparable conditions, the operating parameters for the aeration tanks and final clarifiers were monitored. The average values for these parameters during the study are shown in Table 3.

Table 3: Average Operating Parameters of Plant 1 and Extension 1

Parameter	Plant 1(control)	Extension 1 (Test)	
	Aer.Tank 1,2	Aer. Tank 3	Aer. Tank 4
Average Flow (m^3/d)	24,740	11,250	12,430
Aeration			
Detention Time (hrs.)	7.16	7.46	6.72
Organic Load (kg BOD/m^3d)	0.64	0.64	0.74
MLSS (mg/L)	3,026	2,729	3,256
F/M	0.25	0.29	0.39
SVI	83.0	82.7	86.2
Final Settling	Final 1-4	Final 5	Final 6
Detention Time (hrs.)	3.85	3.82	3.54
Surface Overflow (m^3/m^2.d)	23.91	25.73	
Weir Overflow (m^3/m.d)	87.09	68.26	73.48

It should be noted that Aeration Tank 4 was taken out of service for repairs to be effected to Final Clarifier 6 on day 38 of the study. The values presented reflect the performance of the tank until that point in the study.

Throughout the study, the operation of Extension 1 was carefully examined to determine whether the bio-P system exhibited any operational difficulty. After day 36 of the study, it was observed that some odour could be detected in Aeration Tank 3. By day 38, the operating configuration of Aeration Tank 3 was changed to that of Aeration Tank 4 (one mixer in the first one third of the first pass and reduced air flow to the next one third). This was the same day that Aeration Tank 4 was taken out of service for repair to Final Clarifier 6.

The sludge volume indexes during the study for the control and test tanks are shown in Figure 4. With the exception of two high values for Aeration Tank 4, the SVI's recorded for the test tanks were generally below 120 and comparable to the control section. The high values were recorded within the first 10 days of the study and can be attributed to a period of acclimation. No problems were encountered with the final clarifier operation of the test tanks in terms of sludge bulking.

The mixed liquor suspended solids shown in Figure 5 are more variable, particularly for the test tanks. However, the average MLSS values and F/M ratios shown in Table 3 indicate that the overall loading on the aeration

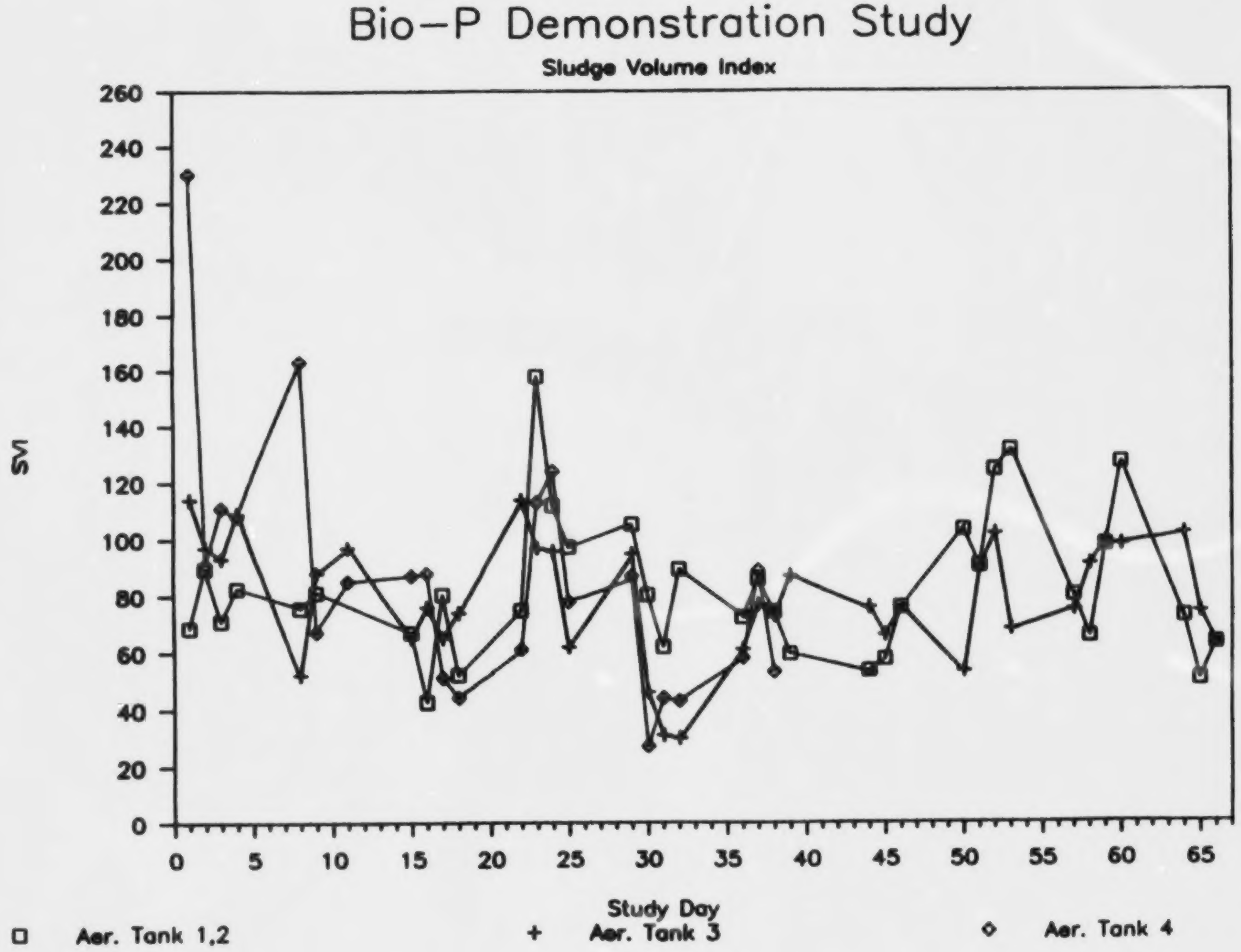

Bio-P Demonstration Study
Sludge Volume Index
SVI
260
240
220
200
180
160
140
120
100
80
60
40
20
0
0
5
10
15
20
25
30
35
40
45
50
55
60
65
Study Day
□ Aer. Tank 1,2
+ Aer. Tank 3
◇ Aer. Tank 4

FIGURE 4

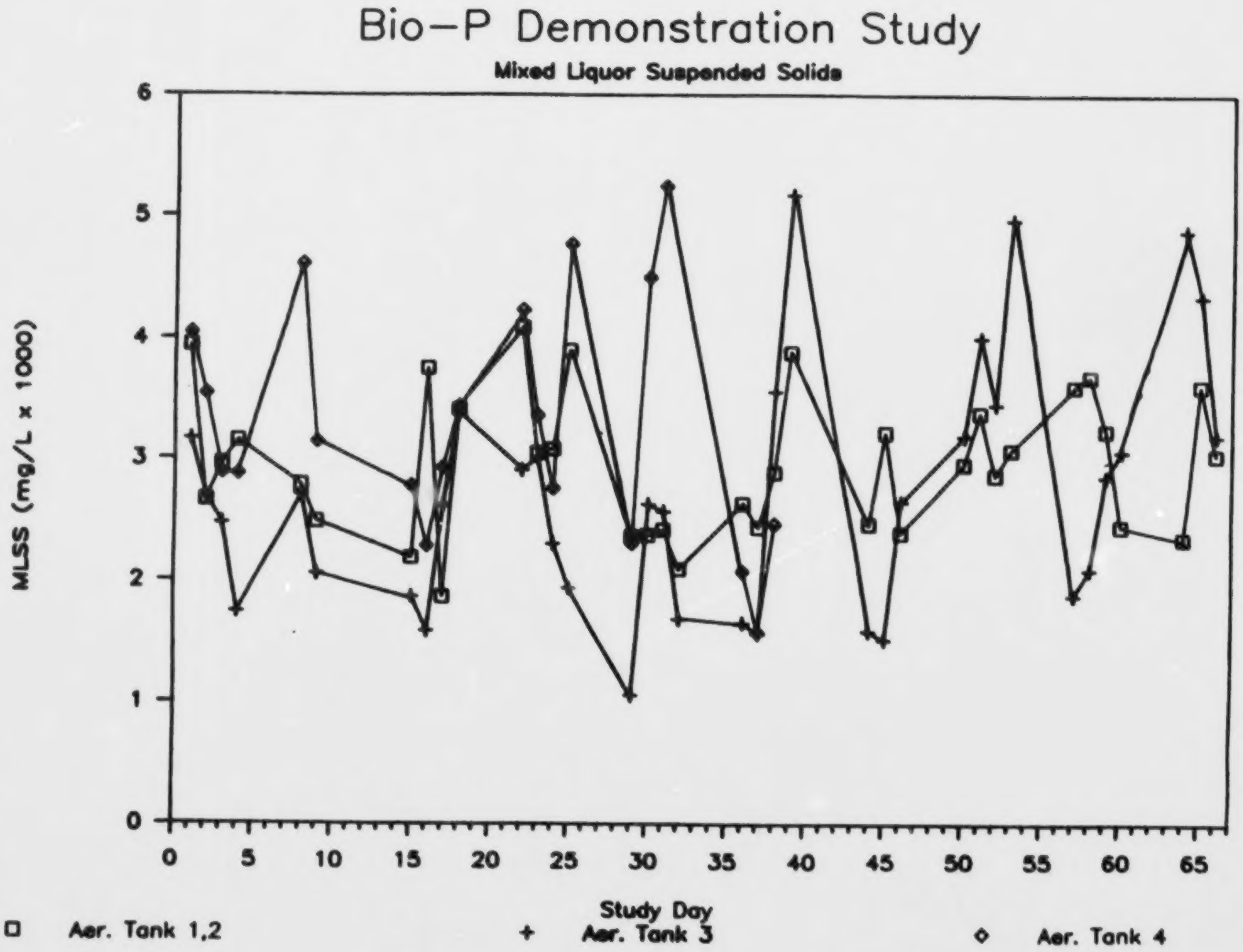

Bio-P Demonstration Study
Mixed Liquor Suspended Solids
MLSS (mg/L x 1000)
6
5
4
3
2
1
0
0
5
10
15
20
25
30
35
40
45
50
55
60
65
Study Day
□ Aer. Tank 1,2
+ Aer. Tank 3
◇ Aer. Tank 4

FIGURE 5

tanks was similar. The average F/M for aeration tank 4 was slightly higher because it was taken out of service during a high flow period.

Similar levels of treatment were obtained with the control and test tanks. This is illustrated in Figures 6 through 8 which depict the effluent BOD, SS, and P trends for the duration of the study. The average values for these parameters are listed in Table 4. The effluent BOD requirements imposed by the Ontario Ministry of the Environment were exceeded to a small degree by Plant 1 (the control system). However, the average effluent P concentration for both plants met the Ministry criterion of 1 mg/L total Phosphorus.

Table 4: Average Effluent Parameters

Parameter	Plant (Control)	Extension 1 (Test)	
	Final 1 - 4	Final 5	Final 6
BOD (total)	25.2	17.5	17.2
SS	21.5	14.6	23.3
P (total)	0.9	1.0	0.6

While a fluctuation in the effluent quality can be expected in any biological system, there were several incidents which contributed to the variation seen in Figures 6 to 8. On day 17 of the study a break in a sewer line near the Etobicoke Creek resulted in increased flow through infiltration to the entire plant. At the same time, a final clarifier in another section of the plant was taken out of service for repair. A portion of the flow from this section had to be directed towards Plant 1 and Expansion 1. An immediate rise in the effluent BOD concentration can be seen in Figure 6 on the second day after this occurrence. Similarly, the effluent SS and P concentrations jumped, although this took an additional two days to become evident. The sewer line was repaired by day 24 and the final clarifier was put back into service on day 26. A reduction in all three of the parameters monitored is apparent at day 28 of Figures 6 to 8.

Discharges from nearby industries were observed to enter the plant on several occasions. Specifically, these discharges were recorded on days 10, 18 to 20, 27, 39 and 44 of the study. The impact on the plant performance was apparent once the flow was reduced to normal levels. The discharge on day 39 is followed by an increase in the plant effluent parameters on day 43. After a short period of stabilization, the discharge of day 44 can be identified by another increase in the effluent parameters.

The flow and the organic load rate to the control and test systems are shown in Figures 9 and 10 respectively.

Anaerobic Phosphorous Release Tests

The results of a preliminary study had indicated that there were large concentrations of residual iron entering the system through the Hyan effluent and sludge thickener sidestream. To determine the concentration of iron

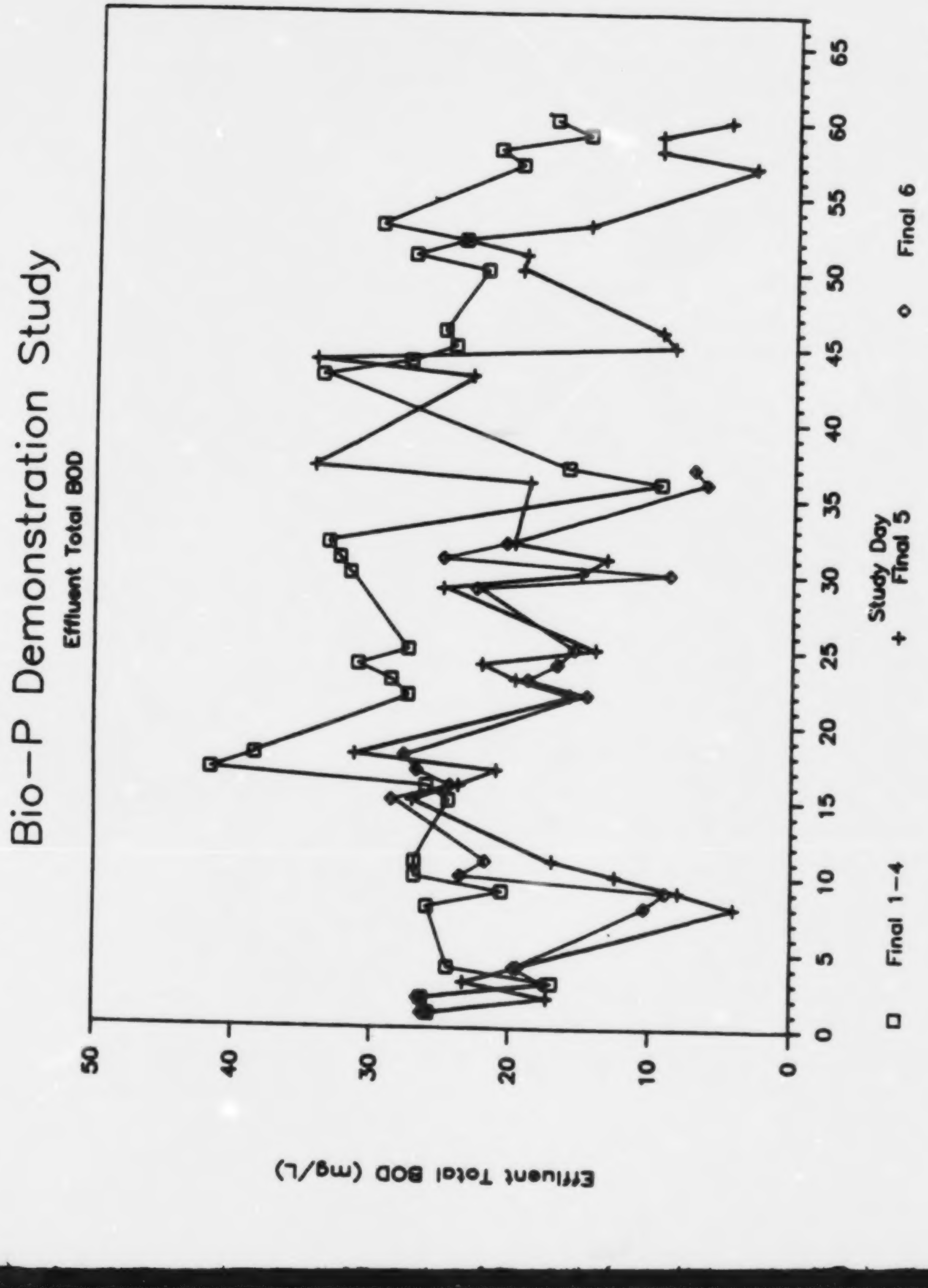

Bio-P Demonstration Study
Effluent Total BOD
Effluent Total BOD (mg/L)
0
10
20
30
40
50
0
5
10
15
20
25
30
35
40
45
50
55
60
65
Study Day
□ Final 1-4
+ Final 5
◇ Final 6

FIGURE 6

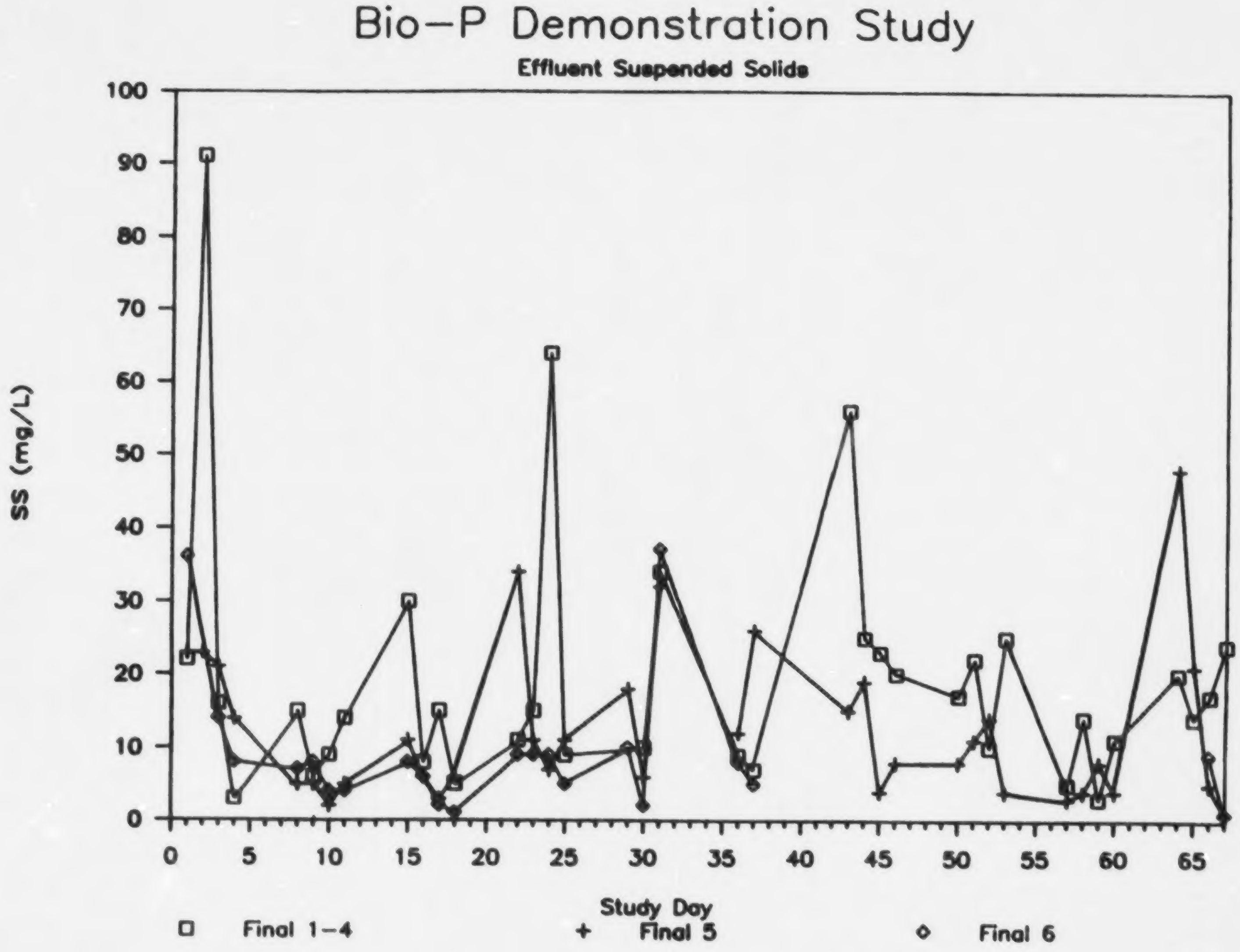

Bio-P Demonstration Study
Effluent Suspended Solids
SS (mg/L)
100
90
80
70
60
50
40
30
20
10
0
0
5
10
15
20
25
30
35
40
45
50
55
60
65
Study Day
Final 1-4
Final 5
Final 6

FIGURE 7

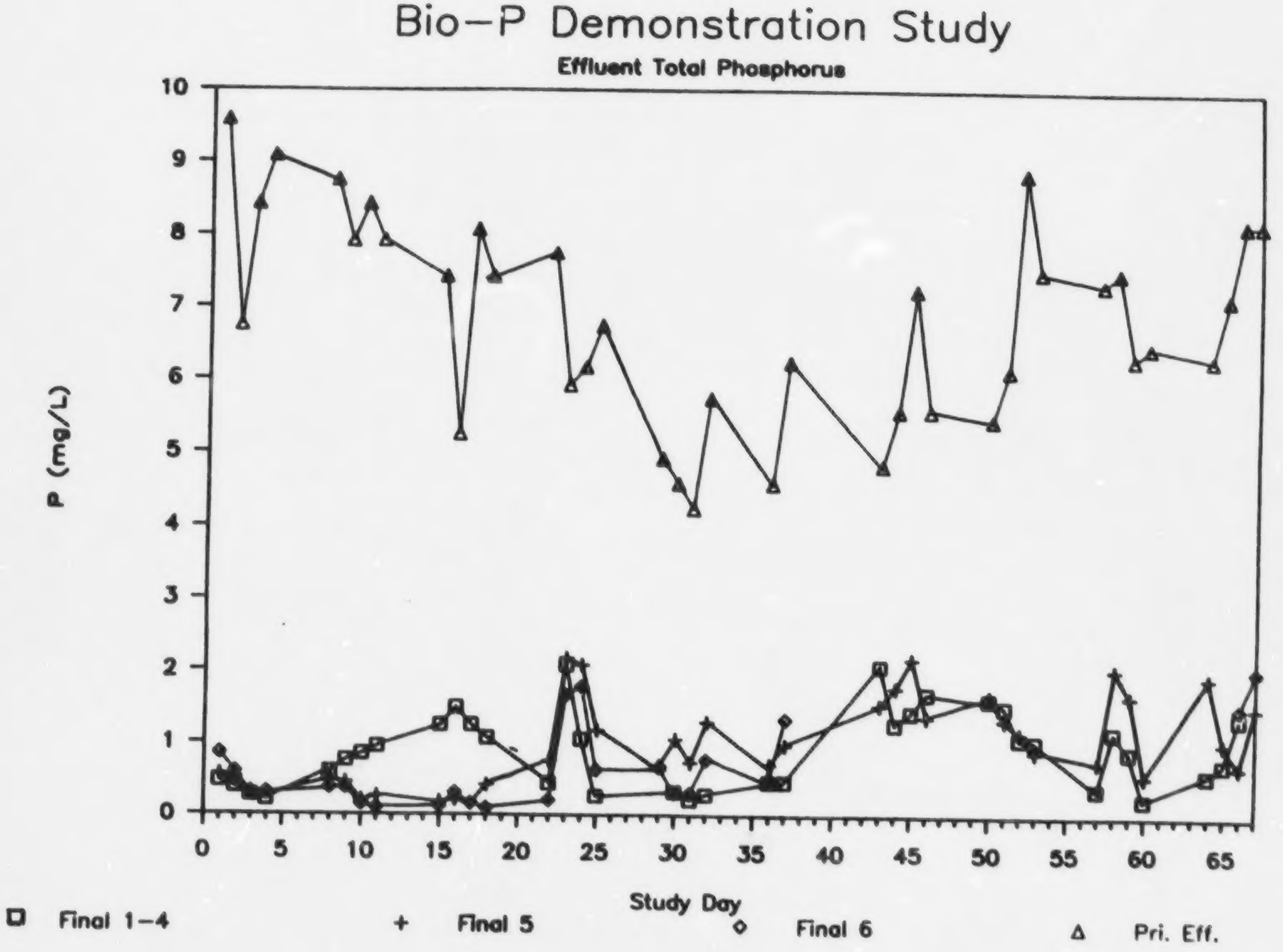

Bio-P Demonstration Study
Effluent Total Phosphorus
P (mg/L)
10
9
8
7
6
5
4
3
2
1
0
0
5
10
15
20
25
30
35
40
45
50
55
60
65
Study Day
□ Final 1-4
+ Final 5
◇ Final 6
Δ Pri. Eff.

FIGURE 8

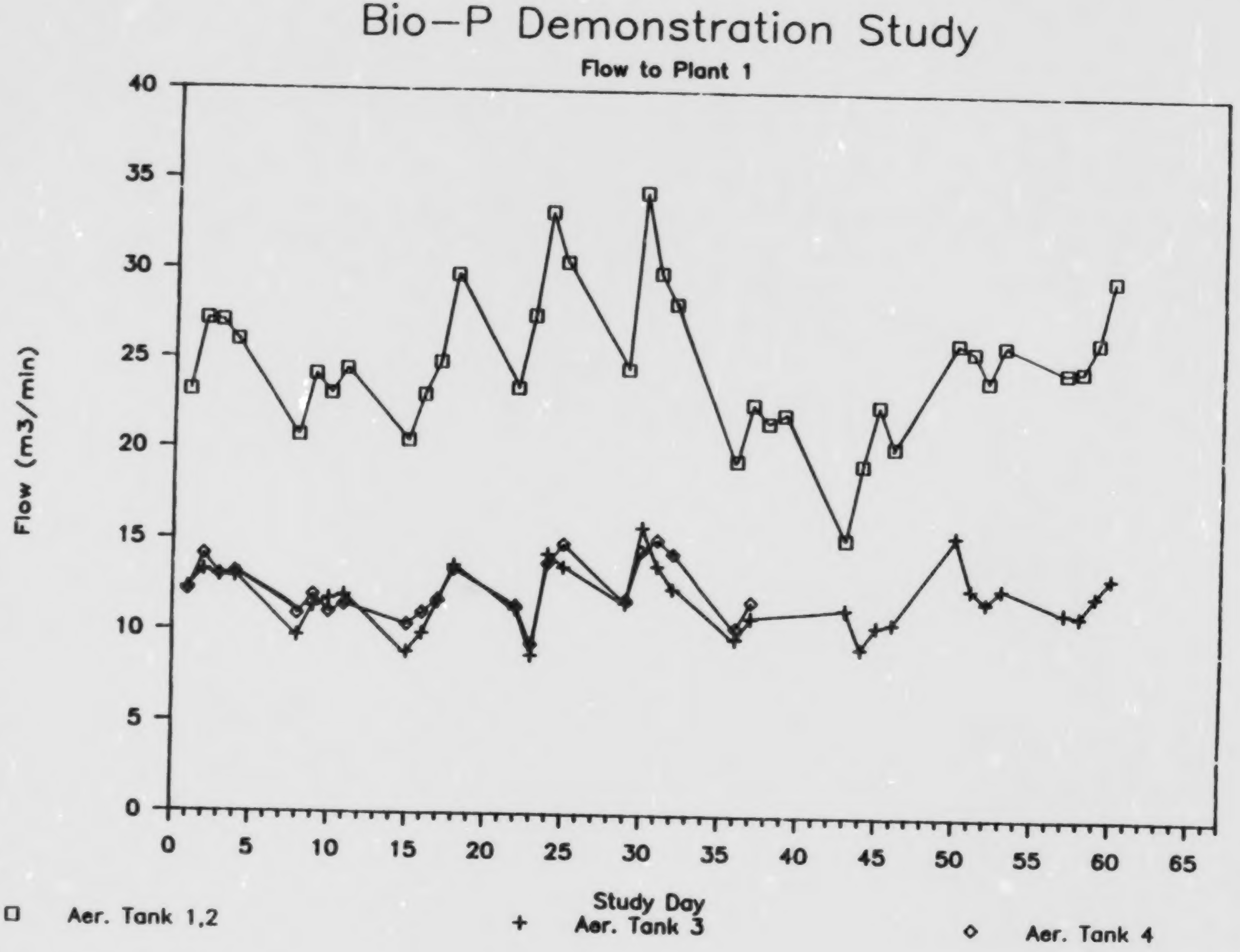

Bio-P Demonstration Study
Flow to Plant 1
Flow (m3/min)
0
5
10
15
20
25
30
35
40
Study Day
0
5
10
15
20
25
30
35
40
45
50
55
60
65
□ Aer. Tank 1,2
+ Aer. Tank 3
◇ Aer. Tank 4

FIGURE 9

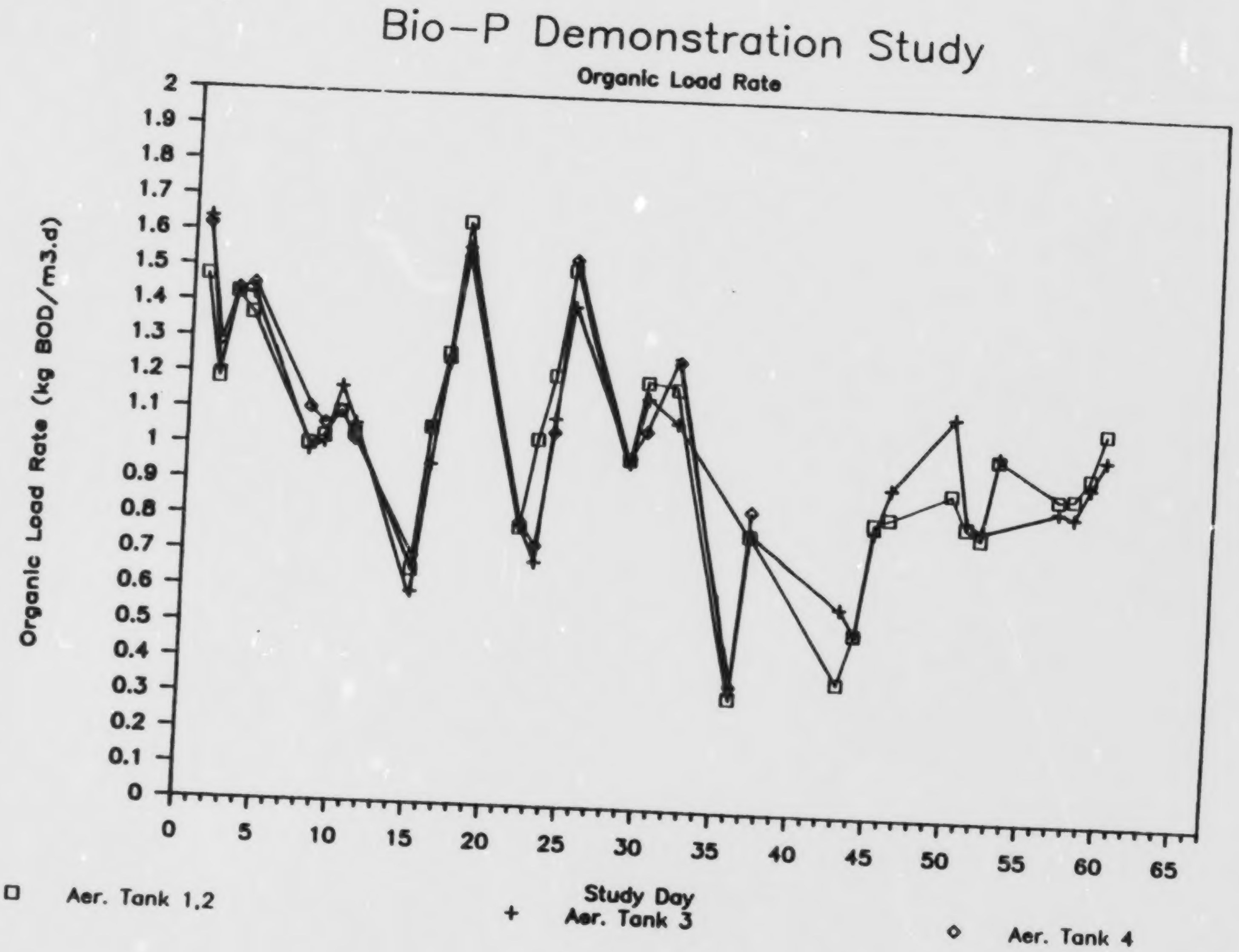

Bio-P Demonstration Study
Organic Load Rate
Organic Load Rate (kg BOD/m3.d)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2
Study Day
0
5
10
15
20
25
30
35
40
45
50
55
60
65
□ Aer. Tank 1,2
+ Aer. Tank 3
◇ Aer. Tank 4

FIGURE 10

entering and leaving the bio-P systems, iron samples were taken of the raw sewage, primary effluent, and the effluent from each system. The results of the analyses on these samples are shown in Table 5.

Table 5: Iron Concentrations at Various Locations in Plant 1 and Extension 1

Sampling Location	Fe Concentration (mg/L)
Raw Sewage	4.4
Primary Effluent	7.7
Plant 1 (Control)	
Final 1 - 4	1.2
Extension 1 (Test)	
Final 5	0.52
Final 6	0.39

As expected, the iron concentration in the primary effluent was significantly higher than in the raw sewage. This raised the question of whether the low phosphorus levels witnessed in the test system were a result of biological uptake or precipitation with the recycled iron in the system.

Previous research has demonstrated that phosphorus that is precipitated with metal salts is not released under anaerobic conditions[15]. Conversely, phosphorus that has been assimilated for cellular metabolism is released in soluble form. A test program was initiated to determine if the phosphorus in the sludge of the test section was organically or inorganically bound.

Grab samples were taken of the return sludge from Plant 1 and Extension 1. The samples were analyzed for suspended solids, iron, total and soluble phosphorus. They were then capped and allowed to go septic over a 5 day period. After this time, the samples were once again analyzed for soluble phosphorus to measure the quantity released under anaerobic conditions. The average values for these analyses are listed in Table 6.

Table 6: Anaerobic Phosphorous Release Tests

Parameter	Plant 1 (Control) Aeration Tanks 1, 2	Extension 1 (Test) Aeration Tanks 3,4
%VSS	67.2	76.9
VSS (mg/L)	5641 (5943)	4771 (5334)
mg Fe/g VSS	162.6 (155.6)	45.9 (36.4)
mg P/g VSS	47.9 (37.5)	48.0 (35.5)
mg P released/g VSS	0.7 (0.4)	6.2 (26.7)
% P released	1.5 (0.9)	14.4 (55.5)

In each case, the amount of soluble phosphorus released by the bio-P system sludge was greater than that released by the control system sludge. On average, this was 14.4% as compared to 1.5%. The numbers in parentheses are average values obtained for two pairs of samples which were allowed to remain under anaerobic conditions for an additional 7 days. For these samples, a much larger average quantity of phosphorus was released by the sludge in the test system 55.5% as compared to 0.9%.

The iron concentration in the control section sludge was greater than that in the bio-P system, as would be expected. The average concentration of iron in the control system was 162.6 mg Fe/g VSS and 45.9 mg Fe/g VSS in the bio-P test system. The relatively high value for the bio-P test system can be attributed to the recycled flows from the previously mentioned process sidestreams. This iron, however did not have any demonstrable effect on the fate of the phosphorus as was shown by the release of phosphorus under anaerobic conditions. The volatile solids content of the test system sludge was significantly higher than that of the control system because of the reduced iron concentration.

As a further check on the nature of the phosphorus removal in the test system, grab samples of mixed liquor were taken along the length of Aeration Tank 1 and Aeration Tank 3 and analyzed for soluble phosphorus. The resulting phosphorus profile is shown in Figure 11. In accordance with the bio-P removal process described earlier, Aeration Tank 3 exhibited a release of soluble phosphorus in the initial anaerobic section followed by an assimilation of the phosphorus in the aerobic section. In comparison, the soluble phosphorus concentration in Aeration Tank 1 varied only slightly along the length of the tank with reductions occurring in the first section and at the point of chemical application in the end of the second pass.

Benefits of Bio-P Removal

Substantial cost savings may be realized through the implementation of bio-P removal. The greatest savings are associated with reductions in aeration power requirements and chemical precipitant.

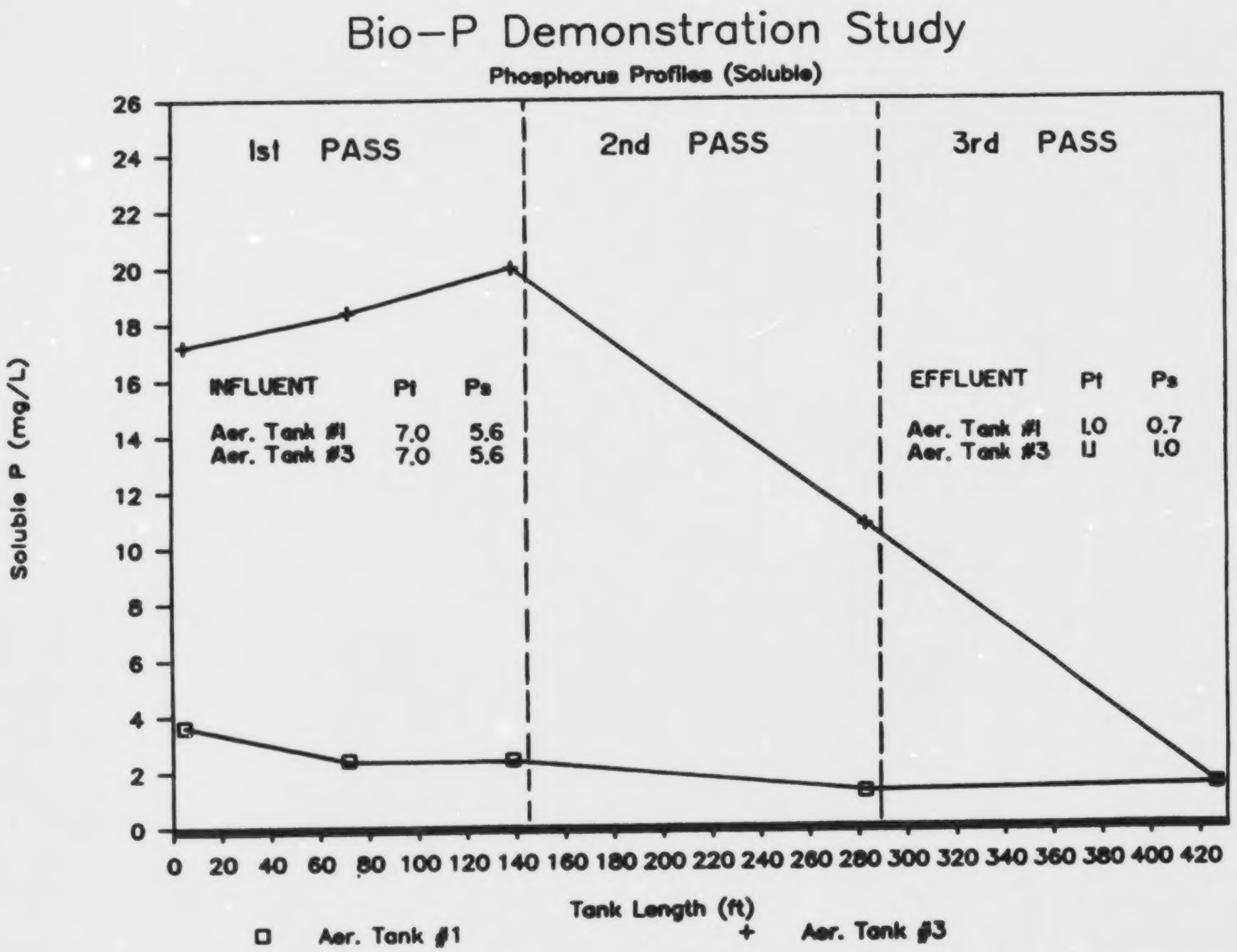

Bio-P Demonstration Study
Phosphorus Profiles (Soluble)
1st PASS
2nd PASS
3rd PASS
INFLUENT Pt Ps
Aer. Tank #1 7.0 5.6
Aer. Tank #3 7.0 5.6
EFFLUENT Pt Ps
Aer. Tank #1 1.0 0.7
Aer. Tank #3 1.1 1.0
Soluble P (mg/L)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
Tank Length (ft)
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420
□ Aer. Tank #1
+ Aer. Tank #3

FIGURE 11

There are a total of 18 valves for the control of air to each aeration tank in Plant 1 and Expansion 1. During the bio-P study, 3 of these valves were closed off completly (to provide an anaerobic zone) and another 3 were adjusted to limit the amount of air in each aeration tank (to provide an anoxic zone). On this basis, it was calculated that a 16% reduction in aeration power requirements is possible with the implementation of bio-P removal.

The total cost for electrical power at the Lakeview WPCP is approximately $1,700,000.00 annually, or $4,650.00 per day. An energy audit conducted on the plant revealed that approximately 51%, or $2,370.00 per day is attributed to aeration power requirements. If it is assumed that a 16% reduction in power cost, then implementation of bio-P will result in a saving of approximately $380.00 per day, or $138,700.00 annually.

Under normal operating conditions, a dose of 10 mg Fe/L in the influent is maintained at the Lakeview WPCP for phosphorous removal. The addition of iron cannot be discontinued completely at the plant because, as with any biological system, upsets may occur. However, if it is assumed that a concentration of 3 mg fe/L is sufficient to maintain the required effluent phosphorus concentration of 1 mg/L during adverse conditions (as was demonstrated in a preliminary study), a 70% reduction in chemical use may be realized. In terms of chemical cost, this represents an annual saving of approximately $70,000.00 for the entire Lakeview WPCP (based upon the 1987 cost of chemical precipitant).

An average increase of 35% in sludge volume may be expected in secondary plants which employ metal salts for phosphorus removal[16]. The quantity of sludge wasted from Plant 1 and Expansion 1 was monitored during the study to see if there was a reduction in sludge production through the implementation of bio-P removal. The monitoring program revealed that, on average, 577.7 m^3/d were wasted from the control system as compared to 494.9 m^3/d from the test section, a difference of 17%.

Bio-P Removal Retrofit at Lakeview WPCP

Based upon the successful results of the bio-P study, it was decided to conduct a detailed review of the Lakeview WPCP to assess the feasibility of a bio-P removal retrofit for the entire plant.

The design data for each of the individual plants was tabulated and compared to the design data for Extension 1. Plant operating records for previous years were also examined to determine the most adverse conditions to which the plant would be subjected.

It was found that the design data for the other plants at the facility were comparable to Extension 1 in terms of the parameters critical to bio-P removal. In addition, it was determined that the extreme operating conditions to which the entire plant might be subjected were encountered at various times during the bio-P study as part of the normal plant operations.

From the data obtained during the bio-P study, a capital cost was developed for a retrofit for the entire plant. This was estimated to be approximately $520,000.00. The payback period, for a bio-P retrofit using the annual cost savings presented previously was estimated to be 2.5 years.

Acknowledgement

The authors wish to express their appreciation to the management, operations, and laboratory staff of the Lakeview WPCP for their support in this study.

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BIO-DENITRO and BIO-DENIPHO systems. Experiences and advanced model development.

The Danish Systems for Biological N and P removal.

Krüger

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1 BIO-DENITRO and BIO-DENIPHO

Sewage treatment with removal of nitrogen and phosphorus was first taken into consideration as early as 1970.
Through an intensive research effort on the part of the Technical University of Denmark as well as Krüger, these systems were developed to cope with the very low effluent standards for nitrogen and phosphorus.

Until now, more than 30 plants have been constructed in Denmark with nitrogen or phosphorus removal, and this means that there is today a large amount of operating experience from these plants.

The newly set effluent standards are 15 mg BOD/l, 6-8 mg Total-N/l and 0.5-1.5 mg Total-P/l.

These low effluent standards demand a high degree of process control and by means of mathematical modelling it is possible to optimize the biological systems.

The BIO-DENITRO and BIO-DENIPHO nutrient removal processes are performed in phased isolation ditches which are continuous flow, activated sludge systems with phased or intermittent operations.

The BIO-DENITRO and BIO-DENIPHO processes for biological nitrogen removal and biological nitrogen and phosphorus removal, respectively, are shown on the process flow diagrams in fig. 1. Only half of the operational cycle for the BIO-DENIPHO process is shown.

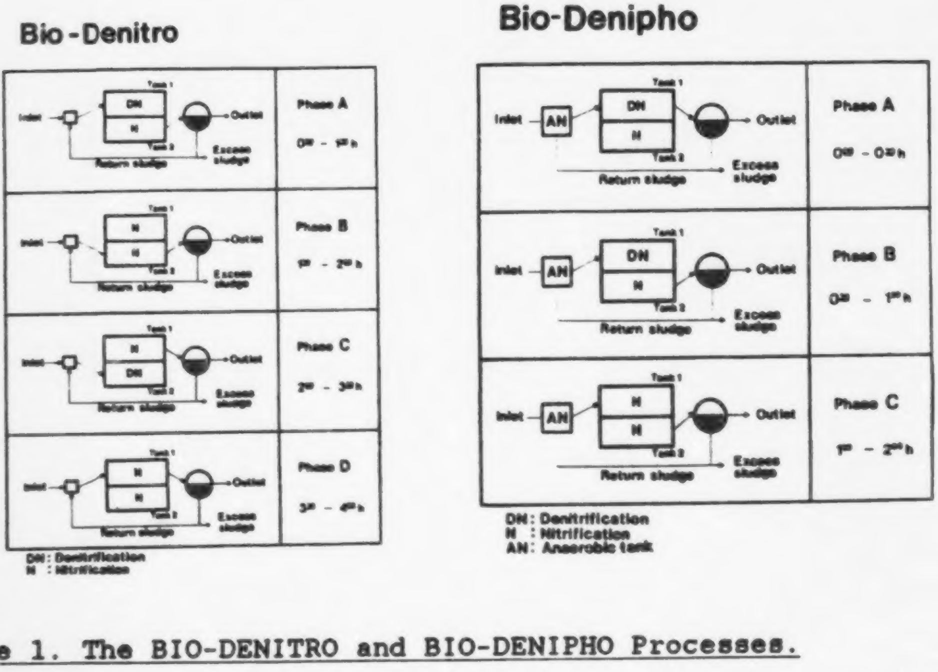

Bio-Denitro
Inlet
Tank 1
Tank 2
DN
N
Outlet
Excess sludge
Return sludge
Phase A
Phase B
Phase C
Phase D
DN: Denitrification
N : Nitrification
Bio-Denipho
AN
Phase A
Phase B
Phase C
DN: Denitrification
N : Nitrification
AN: Anaerobic tank

Figure 1. The BIO-DENITRO and BIO-DENIPHO Processes.

The phased operations of denitrification and nitrification performed in the same ditch give a very flexible biological system which is operated without the need for an internal recirculation pump.

2 OPERATING RESULTS - PILOT SCALE

Table 1 shows the operating results of one BIO-DENITRO and two BIO-DENIPHO pilot plant studies.

The AVEDØRE and KOLDING plants treat settled domestic wastewater while the FREDERICIA plant treats combined domestic and organic industrial wastewater.

Plant	Avedøre (Bio-Denitro)		Kolding (Bio-Denipho)		Fredericia (Bio-Denipho)	
Parameter	Inlet mg/l	Outlet mg/l	Inlet mg/l	Outlet mg/l	Inlet mg/l	Outlet mg/l
Total COD	270	60	260	35	495	99
Dis.COD	208	-	130	-	334	-
Total BOD	100	9	125	9.4	305	9.8
Dis.BOD	73	-	73	-	220	-
Total N	31	5.3	29	6.8	28	3.5
Ammonia N	20	1.4	-	1.2	13	1.2
Nitrate N	1.5	2.4	1.1	4.5	<1	1.4
Total P	8.1	0.6*	7.7	3.9 (<1)**	10	1.6
Phosphate P	6.5	0.4	-	3.6	7.0	1.3
SS	90	<5	100	6	138	12
BOD/N	3.2		4.3		11.1	
BOD/P	12.3		16.2		30.5	

* Addition of 15 mg Fe/l, ** Addition of 6 mg Fe/l

Table 1: Pilot-Scale Results - AVEDØRE, KOLDING and FREDERICIA.

3 FULL SCALE OPERATION

The first Danish full-scale BIO-DENIPHO plant designed with biological phosphorus removal is the NØRRE ÅBY plant that treats combined domestic and industrial wastewater which gives very favourable BOD/N and BOD/P ratios.

The plant is designed for 12,500 PE corresponding to a dry weather flow of 3,200 m3/day and was put into operation at the end of 1985.

Operating results with and without chemical addition (contact filtration with addition of ferric chloride) are given in Table 2.

Parameter	Inlet	Outlet*)	Outlet**)
BOD	234	8.8	5.5 mg/l
Total N	25	4.7	3.4 mg/l
Ammonia N	10	1.0	1.0 mg/l
Nitrate N	-	2.9	2.0 mg/l
Total P	11	3.7	0.85 mg/l
Phosphate P	-	3.3	0.55 mg/l
SS	270	<10	<10 mg/l

BOD/N = 9.4 BOD/P = 21.3

*): Running in of the Plant. No ferric dosing.

**): April-October 1987. Ferric dosing. Me/P: 0.25

Table 2: BIO-DENIPHO Full-Scale Results. The NØRRE ÅBY Plant.

3 Numerical models for activated sludge sewage treatment plants

The IAWPRC has developed numerical steady state and dynamic models, which can be implemented on a normal size PC. The models can predict the fate of organic matter (COD) and nitrogen compounds in activated sludge systems. (Ref.: 1, 2).
These models have been extended also to include the processes of biological phosphorus removal. (Ref.:3).

The biological processes included in the IAWPRC model are:

- aerobic/anoxic growth of heterotrophic bacteria
- aerobic growth of autotrophic bacteria
- "Decay" (death) of autotrophic/heterotrophic bacteria
- Ammonification of soluble organic nitrogen
- Hydrolysis of organic matter and organic nitrogen

In the extended model version, the following processes to model biological phosphorus are incorporated:

- Aerobic/anoxic growth of phosphorus accumulating bacteria
- "Decay" (death) of phosphorus accumulating bacteria
- Anaerobic assimilation of organic matter by phoshorus accumulating bacteria
- Aerobic assimilation of phosphorus by phosphorus accumulating bacteria
- Release of phosphorus by death of phosphorus accumulating bacteria

In the extended model, degradable organic matter has been divided into the fractions easily and slowly degradable soluble organic matter in addition to particular, particular inert and soluble inert organic matter. The IAWPRC-model hereby includes 8 biological processes, the extended model 17 processes.

As will be seen later, simulation results from these models have shown good agreement with experimental data.

4 Model calibration

As described, the numerical models include several expressions for the biological processes assumed to take place in an activated sludge treatment plant. Hereby the demand for knowledge of various "process constants", such as yield factors, half-saturation constants and growth factors, is essential.

It is not possible to develop an overall universal model, that is a model which can be used for different treatment plants (and hereby different types of wastewater) without any form of plant specific calibration.

The former described BIO-DENITRO and BIO-DENIPHO treatment plants' cyclic variations of nitrogen and phosphorus offer a good basis for a model calibration. In connection with the IAWPRC model, the Association gave a range within which the above process constants could be assumed to lie. Using these values in the model and the final calibrated models, simulation results are compared with experimental data obtained from the AVEDØRE BIO-DENITRO pilot plant, fig.2.

In relation to the IAWPRC's "standard values", the calibration showed that the half-saturation constants for ammonia and oxygen had to be reduced as the only parameters. As can be seen, the calibrated model shows very good agreement with the experimental data.

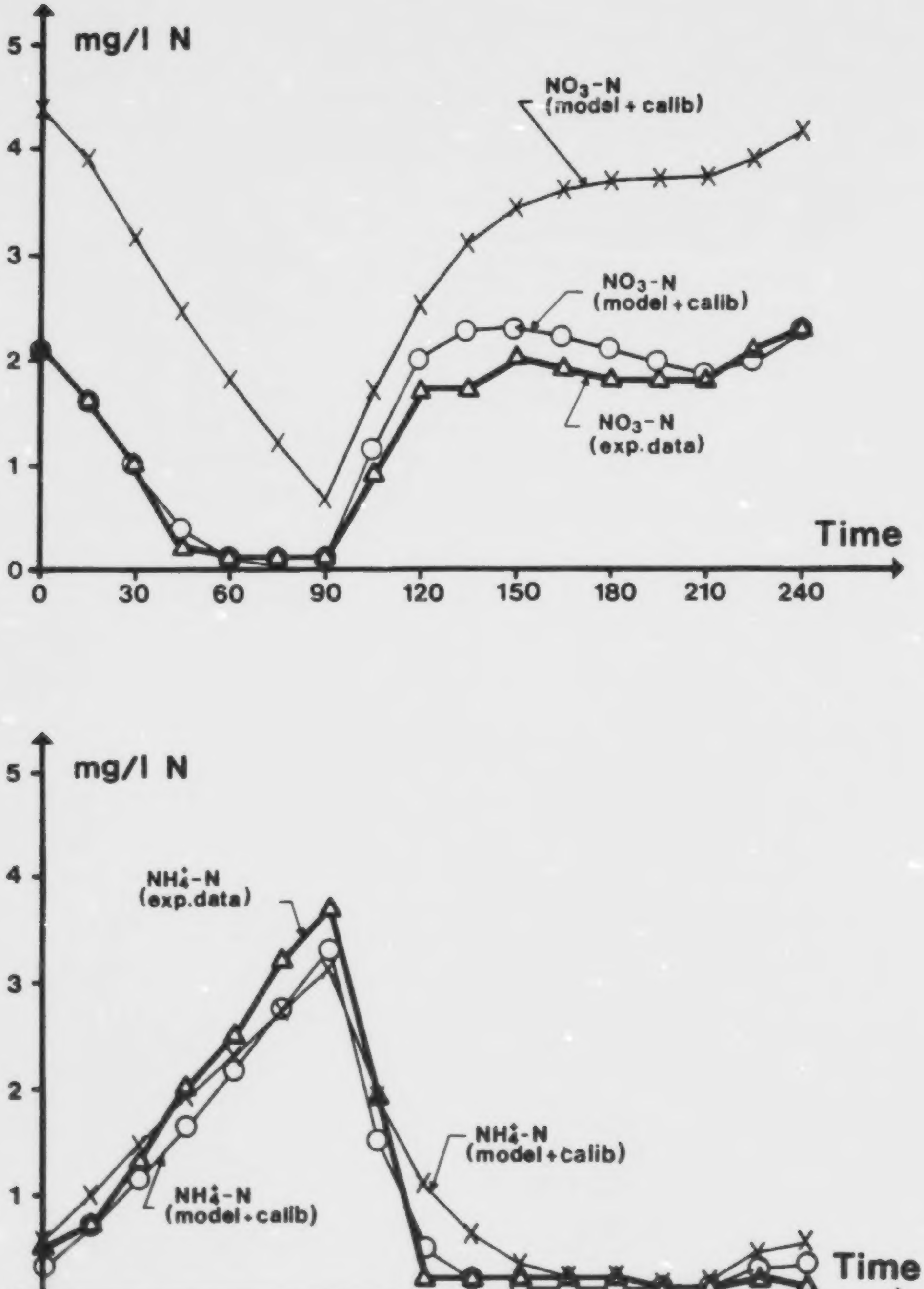


Fig. 2 Experimental data from AVEDØRE BIO-DENITRO pilot plant compared with model-predicted results (calibrated and non-calibrated)

5 Use of numerical models

One of the main advantages of using numerical models of activated sludge treatment plants is the possibility of comparing various designs of treatment plants in relation to nutrient removal.

The following calculations have been made on the basis of two systems:

1. The BIO-DENITRO system
2. The Recirculation system

The calculations have been made for the same plant volume (20,000 m3 activated sludge volume), and the same wastewater composition.

The plant configurations are shown below.

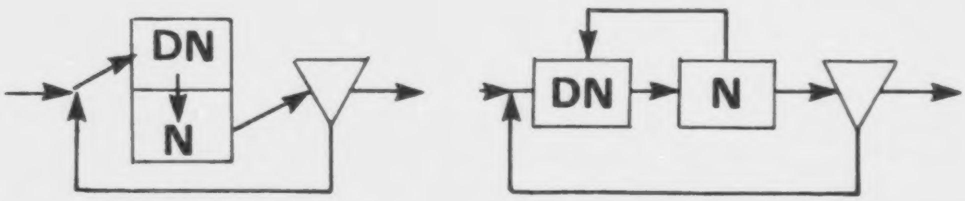
The supplementary conditions for the calculated results were the following:

Oxygen level in the activated sludge was 0 or 1 mg/l during denitrification and nitrification periods, respectively.

Kinetic parameters and the wastewater composition are chosen from the reference values in the IAWPRC model manual (Ref.: 1) corresponding to Danish conditions.

The removal efficiency of the Recirculation system is dependent on the internal recirculation rate from the nitrification tank back to the denitrification tank Q_r/Q.

BIO-DENITRO system   Recirculation system



Assumptions:	Wastewater flow	25,000 m3/day
	Total COD	500 mg/l
	Degradable COD	360 mg/l
	Total N	51 mg/l
	NH3-N	30 mg/l
	NO3-N	1 mg/l
	Recirculation from clarifier	50%
	Temperature of wastewater	10°C
	Hydraulic retention time	19 h
	Sludge load approx.:	0.16 kg COD/kg SS d

The removal efficiency of the BIO-DENITRO system is dependent on the phase length of the phases B and D (see fig. 1), which are controlling the denitrification length of the cycle.

The calculated effluent values for ammonia and nitrate are presented in table 2.

BIO-DENITRO system			Recirculation system		
Phase B/D min.	NH4+NO3-N mg N/l	NH4/NO3 %	NH4+NO3-N mg N/l	NH4/NO3 %	Q_r/Q -
30	3.7	14	6.7	48	4
15	3.5	20	6.5	51	5
0	3.4	35	5.8	55	6
			5.6	57	7

Table 2. Comparison of nitrogen removal efficiencies of the BIO-DENITRO and the Recirculation systems.

It can be concluded from the calculated effluent values of nitrogen compounds that, at the given assumptions, the BIO-DENITRO system produces an effluent which is at least comparable to that of the Recirculation system.

The BIO-DENITRO system is also much more flexible concerning the treatment of wastewater which changes COD/N ratios.

The BIO-DENITRO system can be adapted to a wide range of wastewater compositions merely by changing the B- and D- phases to have the right Denitrification/Nitrification ratio.

References:

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The Measurement of the Rate of Oxygen Transfer in Clean Water by Computer and Related Limitations

W.C. Belschner
Consulting Engineer

One of the main thrusts of research in the field of oxygen transfer is to conserve energy. Until about 1974, consulting engineers throughout the world formulated plans for water aeration systems based on the results and data on aeration trials performed by manufacturers of equipment. Most of the tests were performed in basins located on the property of the suppliers. Although these aeration tests were conducted on water devoid of oxygen and at about twenty degrees centigrade, research has established that apparently small or subtle differences in the methods of data interpretation can result in as much as a 10 % change in the clean water standard oxygen transfer rate Such a difference can make or fail a transfer rate test requirement.

The writer was involved in Montana in 1981 in a court case where the difference in a required transfer efficiency rate of 10 % was not within 5 % of this amount and the system was refused by the Consultant, Black and Veatch, from Kansas City, Kansas.

At this writing several aeration equipment manufacturers are using a wide variety of techniques which result in substantial variations in data for the same aeration device. As a consequence, consulting engineers now write requirements for field testing in full scale basins into their specification. As a result, contractors and manufacturers recognize this liability or risk and in preparing bids will include larger factors of safety. So many aerations will be adequate but over designed.

A Committee on Aeration was formed in the mid-seventies and in 1978 a workshop presented a document compiled by experts across the U.S.A. This document was sponsored by E.P.A. and published in 1979. It is called " Workshop Toward an Oxygen Transfer Standard ". The use of computers became more widespread in this field as one of the results of the workshop and the same Committee continued its work with E.P.A. producing in 1983 the publication " Development of Standard Procedures for Evaluating Oxygen Transfer Devices " EPA-600/2-83-102, 209 (Oct 1983). This was promptly followed in July 1984 by the now present " A.S.C.E. Standard Measurement of Oxygen Transfer in Clean Water."

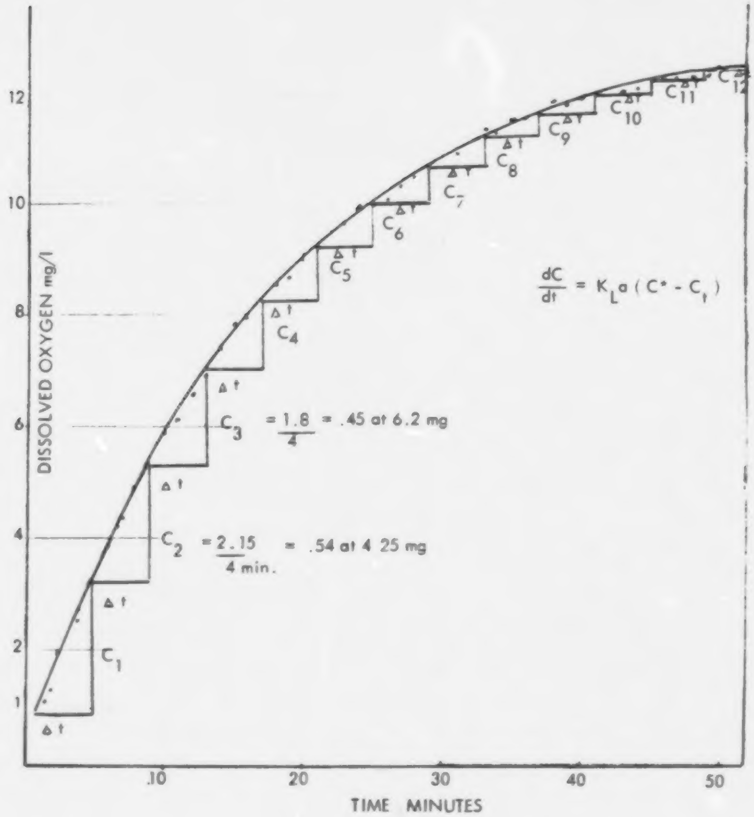
Until about 1981, most engineers specified a transfer testing methodology using the oxygen value measured at the end of the trial in the aeration basin as the saturation value to calculate the mass oxygen transfer rate (SOTR). The apparent oxygen transfer coefficient K_La was calculated by the well-known log deficit equation and using a least squares linear regression method. The plot of the log deficit was generally a straight line and linear regression yielded an approximate, but generally used result.

The present adopted standard of A.S.C.E. 1984 utilizes a computer program which calculates the apparent volumetric mass transfer coefficient K_La and the equilibrium oxygen concentration C_s at each measured point during the re-aeration or re-oxygenation period. Non-linear regression methodology is used since the direct plot of the data points at each interval is not linear. The computer program then averages all of the calculated values and performs several iterations to arrive at the optimum values of these two above parameters.

The direct plot of a typical re-oxygenation trial is shown figure 1. In order to reduce this curve to allow linear regression computation a number of $\frac{dC}{dt}$ units have been drawn. In this case a unit of four (4) minutes was used as the Δt, to demonstrate the methodology more clearly. If one minute intervals were used, then the approach to linearity would be almost complete. For our example, the regression analysis yields the intercept value on the Y axis as well as the slope ($K_L a$ / minute) value. A correlation value of .994 shows the accuracy Using this data allows us to plot the $\frac{dC}{dt}$ versus dissolved oxygen.

FIGURE 1 TEST No 5 TENNESSEE U.S.A. FEB. 2, 1988

DIRECT DATA PLOT



The actual plot of $\frac{dC}{dt}$ values as calculated from the previous direct plot versus dissolved oxygen, is shown in figure 2 . It is only necessary to locate the intercept found in the regression and using the slope ($K_L a$ / minute) value, the resulting X axis intercept yields the oxygen saturation value C^* .

Scatter at the start of the plot for $\frac{dC}{dt}$ values can be eliminated by using the integrated or exponential values of $\frac{dC}{dt}$, or C versus t. This is the A.S.C.E. procedure for the computer program yielding K_La and C*. The writer has used the graphic method and linear regression to check the computer program results. Based on some fifteen (15) re-aeration trials, the degree of correlation is shown to be about 1 % - 2 % for both K_La and C*. The graphic linear regression calculation is of course more time consuming.

OXYGEN TRANSFER TESTS - General Comments

Text books and data used by both Consulting engineers and manufacturers was based on data formulated by such important researchers as Downing, Eckenfelder, King etc. but this is over twenty years ago. The writer supports the statements made by Professor William C. Boyle of the University of Wisconsin in the March 1988 issue of the Water Pollution Control Federation namely that of the approximately 1.75 million horsepower of aeration equipment currently in place in North America, evidence suggests the oxygen transfer efficiency is low, and power costs could be reduced by up to 50 % by improved design and operation. One basic reason for this situation has been the unavailability of, or failure to use optimal methods for the measurement of Oxygen Transfer.

The redundancy in some of the above statements is purposeful. The upgrading of technology on oxygen transfer has been largely done by Americans and Europeans. It is one thing to specify say a 10 % efficiency of oxygen transfer for an aeration system. This percentage is based on the pounds of air required to degrade the Biochemical Oxygen Demand (BOD) of a particular waste. In Quebec, the Ministry of the Environment asks for two (2) pounds of oxygen per pound of BOD. Many American State criteria vary from one pound to one and a half (1 - 1½) pounds oxygen per pound of BOD.

To emphasize the point, to the writer's knowledge, many of the sewage treatment plants built recently in Quebec were designed on the theoretical basis of 100 mg/liter of BOD whereas in fact repeated wastewater sampling had shown the range of BOD to be from 30-70 mg/l. If then the average BOD down the sewer pipe is 50 - 60 mg/l and the design is made for 100 mg/l or about 80 - 100 % increase and the design parameter used is two (2) pounds of oxygen per pound of BOD whereas the actual theoretical requirement by mass balance is about 1.4 pounds of oxygen per pound of BOD, then a large factor of safety is indeed incorporated in the design. Again as stated in the opening paragraphs, the Contractor or equipment supplier will, in order to meet a 10 % efficiency oxygen transfer, probably add an additional factor of safety in his equipment bid.

One of the main reasons for the foregoing large safety factors used in our aeration design for wastewater is the uncertainty of the ratio of oxygen transfer rate in clean water compared to the transfer rate in a certain wastewater. This so called α factor is further discussed in this paper.

However, we now have a probably good standard for the transfer rate in clean water. Nevertheless, in the writer's experience of testing transfer rates in full scale aeration basins, very few Consulting engineers and government employees, in Quebec at least, have had previous experience with aeration systems including oxygen transfer rate tests. One can therefore more easily understand the figures stated of 50 % excess power in existence in aeration systems operating today.

The development of the program came about from closer examination of direct plots of data from re-oxygenation trials. Evaluation of the data is possible from an examination of the basic two-film theory of mass transfer of oxygen, where

$$\frac{dC}{dt} = K_La (C^* - C) \qquad (1)$$

where $\frac{dC}{dt}$ = rate of oxygen transfer mg/l

K_La = overall oxygen transfer coefficient

C^* = dissolved oxygen saturation mg/l

C = dissolved oxygen at time t mg/l

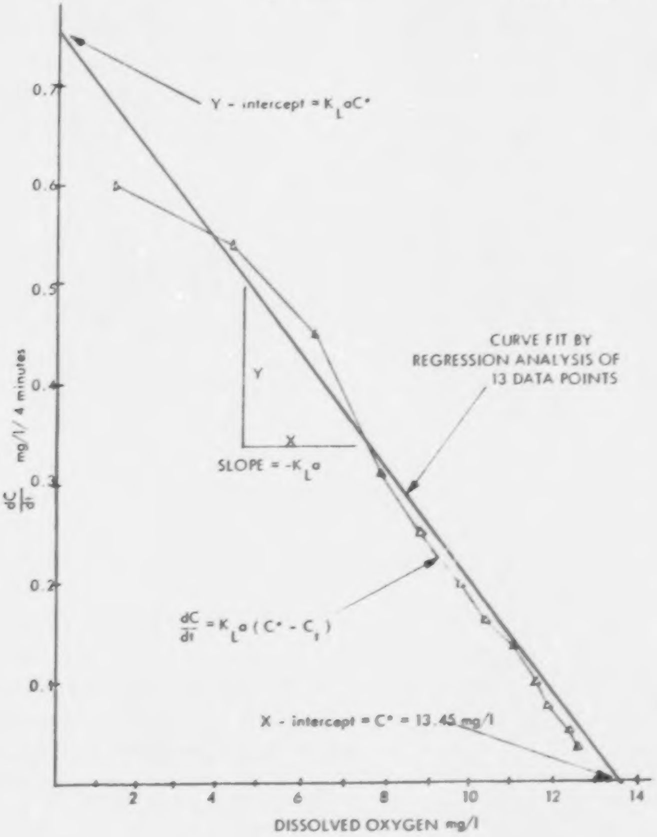
Gilbert and Chen Ref. 2 showed that it is possible to evaluate the data directly without having to assume a value of C^* and K_La, the two (2) main parameters to be determined in any Oxygen Transfer Rate measurement.

Integrating Equation 1, assuming K_La and C^* are constant with respect to time, yields

$$C = C^* (1 - e^{-K_Lat}) \qquad (A.S.C.E.) \qquad (2)$$

This exponential equation mathematically describes the test response curve, C versus t. With the aid of a computer, a least-squares nonlinear regression analysis of the experimental data can be performed to determine the unknown values of K_La and C^*.

FIGURE 2 $\frac{dC}{dt}$ VERSUS C TENNESSEE U.S.A FEB. 2, 1988

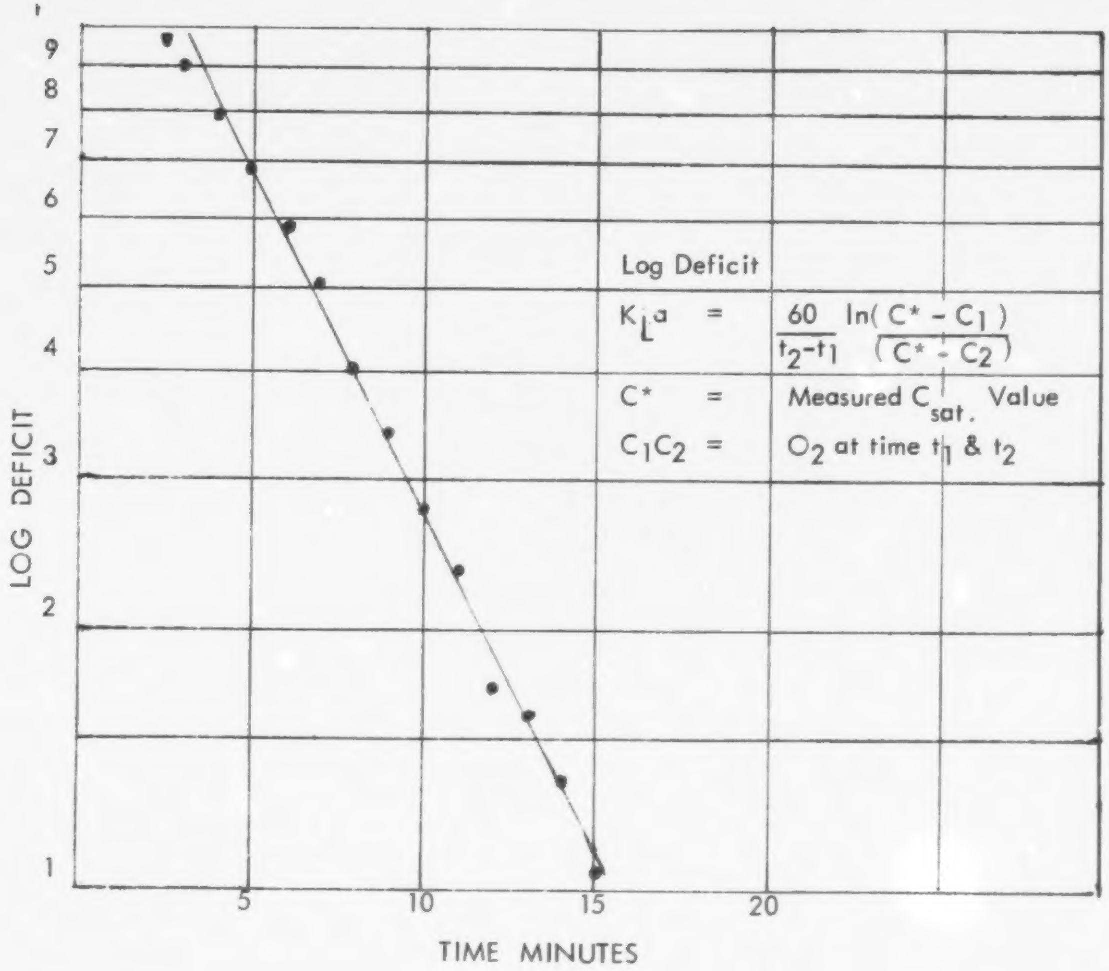


TYPICAL DATA SHEET PRINT-OUT OF C* & K_La/MINUTE

FIGURE 4 TEST No 2 VALLEYFIELD, QUE/86

```
BEST ESTIMATE FOR C-STAR OR USE 10.0 MG/L?? 10.8
BEST ESTIMATE FOR C-ZERO OR USE 0.0 MG/L?? 0
BEST ESTIMATE FOR KLA-PRIME OR USE 4.0 1/HR?? 10.6332

DATA SET FOR TEST NO.VAL2FL
----------------------

ITERATION      C-STAR           C-ZERO             KLA'           SUM OF
   NO.          MG/L             MG/L             1/MIN            SQS
---------      -------          -------           ------          -------

     0          10.800           0.000
1              11.9066         -.264502           .1772          5.8844
2              12.1705         -.35489          .115568           4.81251
3              12.2493         -.348414         .127656           .473025
4              12.2464         -.349426         .126297           .469243
5              12.2466         -.349363         .126385           .469232
6              12.2466         -.349365         .12638            .469233
                                                .12638            .469237
STD. DEVIATIONS OF PARAMETER ESTIMATES

ABS. UNITS : 5.36281E-2        2.19739E-2      1.22036E-3
%OF LSE : .437901             -6.28967         .965624

ESTIMATE OF ERROR = .176869
                  SUMMARY OF DATA
                -----------------
                              D.O.
DATA             TIME     CONCENTRATION    FIT VALUE   RESIDUAL
PAIR              MIN          MG/L           MG/L

 1
 2                .2
 3                .7            .3         -3.49808E-2      .334981
 4                1.2           .725        .717083         7.91693E
 5                1.7           1.35        1.4231         -7.30958E
 6                2.7           1.925       2.08588        -.160876
 7                3.7           3.075       3.29217        -.217168
 8                4.7           4.2         4.35524        -.155241
 9                5.7           5.2         5.29211        -9.21078E
10                6.7           6           6.11775        -.117748
11                7.7           7           6.84537         .154629
12                8.7           7.575       7.48661         8.83942E
13                9.7           8.25        8.05171         .198284
14                10.7          8.65        8.54974         .100262
15                11.7          9.3         8.98863         .311367
16                12.7          9.4         9.37542         2.45819E
17                13.7          9.675       9.71629        -4.12903E
18                14.7          9.95        10.0167        -6.66962E
                  15.7          10.175      10.2814        -.10643
                                10.325      10.5147        -.189739
```

FIGURE 3 TEST No 2 VALLEYFIELD, QUE.
Sept. 30, 1986



From calculated linear regression

K_La = 11.33 at 20° C

$$SOTR = K_La \times C^* \times \frac{C_s}{C_m} \times \frac{BP_a}{BP_t} \times \text{Mass } H_2O$$

$$= 11.33 \times 11.0 \times \frac{9.07}{9.69} \times 0.995 \times 4.96$$

$$= 566.4 \text{ Kg/ Hr.}$$

$$OTE = \frac{566.4 \times 100 \times 2.2}{5440 \times 1.035} = 22\ 13\ \%$$

An attempt to broaden the data base and experience of oxygen transfer testing is therefore made here and some of the main items follow.

a) Since an average test for oxygen transfer testing in a full size aeration basin of say 500,000 imperial gallons (2,270 m^3) is about \$ 10,000. and can vary from perhaps \$ 5,000. - \$ 18,000., the Consulting Engineer and Contractor should know what the figure is for his cost estimate. None of the Consultants or Contractors who engaged the writer to perform oxygen transfer tests, in several municipal plants, had estimated more than \$ 1,000. for this work.

b) Since one oxygen meter and sensor probe of good quality is priced at some \$ 3,000., then a Consultant who requires, in his specifications, the installation of up to twenty (20) meters and sensors is not at all knowledgeable. The use of six (6) meters and probes is probably a maximum, since the purchase, maintenance, calibration and reading of more than this number of units is neither practical nor reasonable.

c) The Contractor or equipment supplier is usually responsible to provide absolutely reliable equipment to measure the air volume rate to the aeration basin. Proper apparatus entails that to measure pressure and temperature of the blower operation as well as an orifice plate, calibrated annubar or mercury manometer in the air piping supply to the aeration basin to be tested. The firm conducting the oxygen transfer trials is not responsible for the foregoing equipment or apparatus.

d) Solubility of the deoxygenating chemical, sodium sulfite, is about 12.5 % by weight in cold water, or about one third that of common salt. Since in a large basin (above 500,000 imperial gallons) tons of sulfite must be dissolved, depending on the number of specified tests then adequate mixing tanks and pumps should be provided by the project Contractor.

e) Temperature and weather can prevent the testing. Mid May to mid November weather is normally suitable. Heavy or medium rainfall makes for nearly impossible conditions. A.S.C.E. states water temperature should be between 10 - 30° C.

f) Inspection of the aeration basin before testing is usually required to assess the location of oxygen sensors, adequacy of air flow measurement equipment and the available space for the operation of the addition of de-oxygenating chemicals.

g) Doubtful or unacceptable testing results may entail the repetition of some or all or the trials. Additional testing also may be requested by the Consulting Engineer or the Ministry of the Environment. All additional testing would be borne by the Contractor

h) Recorders of oxygenation data linked to the oxygen meter have been written into some specifications. Not only would this add a cost of about \$ 2,000. to each oxygen meter, but, the calibration of each recorder at probably each trial, would increase the cost of re-oxygenation tests by 30 %.

LIMITATIONS OR CONSTRAINTS OF A.S.C.E. METHODOLOGY FOR DATA PROCESSING

1. A requirement for a proven computer program. The writer prefers a known data set to be run through the computer before the latest measurements of an oxygen transfer trial are run.
2. The test runs should be carried on to 98 % of saturation in order to optimize the calculated K_La or transfer coefficient value. This can be time consuming in the case of a low aeration intensity or oxidation ditch rotor aeration, for example.
3. A direct plot of all dissolved oxygen points (D.O. points) versus time of run to evaluate consistency, scatter and to obtain a good estimation of C*, which is added to the da-

ta in the program and reduces the number of iterations.

4. Truncation of data is generally only done on inspection of a direct re-oxygenation data plot or some data scatter during a trial. Usually truncation of the first 20 % of saturation value (C^*) does not greatly affect the calculated K_La and C^*.

5. One must question the validity of the re-aeration graphs and curves in existence at the present time. Some equipment suppliers and manufacturers have modified their older curves to comply with A.S.C.E. Standard. One must ascertain the validity of such up-grading. The supplier's trial data should be made available to the Consultant but this presumes the Consultant to be knowledgeable about re-oxygenation curves to evaluate such results. It is still emperative that for a given aeration system with aeration basins, that the transfer coefficient K_La determined in clean water testing is specific to that basin with that particular aeration system configuration. This also includes, of course, the saturation value C^*.

OXYGEN TRANSFER UNDER PROCESS OR OPERATIONAL CONDITIONS

Probably Professor W.C. Boyle, the Editor of EPA Oxygen Transfer Committee publications has amassed the largest amount of data on K_La and C^* (oxygen transfer rate coefficient and oxygen saturation values, respectively) available today. The K_La_c (clean water) coefficient and the Wastewater coefficient K_La_w expressed as K_La_w over K_La_c yields a fraction known as α. Values of α have been found to range from 0.3 - 1.2, so a measured range of K_La_w values can be used and compared with a previously measured K_La_c for an aeration system as a pre-requisite for oxygen transfer approval.

Twenty or more years have elapsed since some initial tracer re-aeration studies were made. Several methods are available at the present time, which can reasonably measure the K_La_w (wastewater) transfer coefficient. To do so, several parameters must be available or measurable. It is beyond the scope of this paper to present more than an indication of process methodology to acquire a reliable K_La_w and C_w^*. It is one thing to measure the K_La_w for a particular wastewater system but one must know reasonably accurately the K_La_c (clean water) for that basin and system. Then perhaps one can assess whether it is possible to modify the existing aeration system so as to upgrade the value of α to approach unity, or in other words to improve the oxygen transfer capability.

The coefficient and rate of oxygen transfer can vary diurnally as well as spatially through a particular aeration system. Thus a Town such as Ithica New York may be cited as an example in the writer's experience. Town sewage quality varies considerably during twenty four (24) hours as well as weekly. Ithica has a large number of industries plus the sewage contribution of Cornell University. Sewage quality or strength and volume vary greatly (4 : 1) over a twenty four (24) hour period. On week-ends, no industrial activity is coupled with a university student exodus, and the week-end flow and strength of waste is comparable to say a Wednesday 3 A.M. flow and quality. Great flexibility is required to optimize operational costs.

From bibliography available, one of the most readily applicable methods of measuring K_La_w and C_w^* in an existing aeration facility is by the non-steady state testing in well mixed aeration basins. It is similar in many respects to the clean water non-steady methodology.

The requirements are :

a) either a power level change availability in the existing aeration equipment or / and preferably

b) the application of hydrogen peroxide (H_2O_2) to boost oxygen content in the wastewater.

Two other important items are that :

1) the incoming sewage should have constant load and

2) the aeration basin have the continued presence of D.O. (dissolved oxygen) greater than 2 mg/l.

Gerry Shell, one of the EPA Oxygen Transfer Committee members of Tennessee, U.S.A. utilizes a forty five (45) gallon drum and oxygen probe technique to obtain an approximation of α in existing wastewater systems. Ref. 13.

Possibly in a few years time, sufficient data will be available so that a proper analysis of a particular wastewater can be used to apply a corresponding α factor which can optimize the design of that specific wastewater aeration system.

COMPARISON DATA - Log deficit versus nonlinear regression (A.S.C.E.)- Example

A. Valleyfield S.T. Plant :

1) Log deficit method yields SOTR = 566.4 Kg.O_2 / Hr.
Oxygen Transfer Efficiency = 22.13 % see FIGURE 3

2) ASCE program 18 points SOTR = 462.7 Kg.O_2 / Hr.
Oxygen Transfer Efficiency = 18.47 % see FIGURE 4

ASCE is 18. % less on Kg / Hr. Transfer
ASCE is 3.6 % less on Transfer Efficiency

B. Farnham Oxidation Ditch (Rotor Aeration)

1) Log deficit method yields SOTR = 1.47 Kg.O_2 / KW
2) ASCE program yields SOTR = 1.7 Kg.O_2 / KW

ASCE is 15.6 % greater than the log deficit calculation.

CONCLUSIONS

A brief attempt has been made to give more information to Consulting Engineers and equipment suppliers based on field experience in oxygen transfer trials in clean water. Evaluation of comparitive oxygen transfer in wastewater will, in the opinion of the writer be based on sufficient data becoming available from continued wastewater testing to incorporate a ratio of $\frac{K_L a \text{ waste}}{K_L a \text{ clean}}$ or α factor into an aeration system design with an optimistic margin of error of perhaps 10 - 15 %.

We are progressing as with the Committee Consensus Standard of A.S.C.E. in clean water, which can possibly be modified in the future Proper and adequate test procedures for the assessment of aeration equipment under process conditions are less developped at this time. The methods available are costly and require fine expertise.

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SEPARATION DES FONCTIONS BRASSAGE-AERATION

Etude de la circulation et du transfert d'oxygène en chenaux d'oxydation

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INTRODUCTION

En France, dans la majorité des stations d'épuration par boues activées, les fonctions brassage et aération sont assurées par un seul et même dispositif, soit du type brosse (axe horizontal), soit du type turbine lente (axe vertical). Le rôle de l'aérateur de surface est alors double :

- créer une circulation suffisante du liquide pour maintenir en suspension les solides,
- assurer un transfert d'oxygène suffisant pour satisfaire la demande en oxygène des boues activées.

La circulation et le transfert d'oxygène sont donc réalisés de façon simultanée par le même système mécanique, d'où un certain nombre d'inconvénients quant à la régulation de l'oxygène dissous sans perturber la circulation.

Ainsi, lors de sous-charge organique où le temps de brassage est supérieur au temps strictement nécessaire à l'aération ou lorsque une nitrification-dénitrification dans un même bassin est demandée, la dissociation des fonctions semble la seule solution économique et fiable.

L'étude présentée concerne les résultats obtenus sur des chenaux d'oxydation à l'échelle laboratoire ($V = 1\ m^3$) et à l'échelle industrielle ($V = 860\ m^3$ et $2800\ m^3$) tant du point de vue circulation (vitesse moyenne de U) que transfert d'oxygène.

La circulation est créée grâce à l'utilisation de mobiles d'agitation à axes horizontaux et l'air est introduit par des systèmes de dispersion placés au fond du chenal.

MATERIEL ET METHODES

Les expérimentations ont été menées sur 3 chenaux de tailles très différentes (1 - 860 - 2800 m³). Les caractéristiques géométriques de ces bassins sont données dans le tableau 1.

Les mobiles d'agitation utilisés sont du type axial. Leurs caractéristiques géométriques sont données dans le tableau 2, ainsi que les valeurs des nombres sans dimension caractérisant leurs performances (nombre de pompage Ngp, nombre de puissance Np).

La mesure de la vitesse d'écoulement a été faite avec un micromoulinet OTT type C2. Sur chaque section de mesure les vitesses sont mesurées en N points répartis (N = 40 sur le chenal 2 - N = 20 sur le chenal 3). La vitesse d'écoulement U correspond à la moyenne des N mesures d'une section. (Aucune précision significative supplémentaire n'a été apportée en faisant le calcul intégrant les surfaces). Sur le chenal 1, la méthode de traçage impulsionnelle avec un électrolyte a été utilisée (injection d'un traceur et suivi de la conductivité en fonction du temps). Elle conduit aux mêmes valeurs de vitesse que le relevé en profil, mais est beaucoup plus rapide à mettre en oeuvre (valable surtout à échelle laboratoire).

La quantité d'oxygène transféré a été calculée à partir de la détermination du coefficient de transfert Kla par la méthode classique de réaération en régime transitoire après avoir désoxygéné l'eau avec du sulfite de sodium (en présence d'ions cobalt).

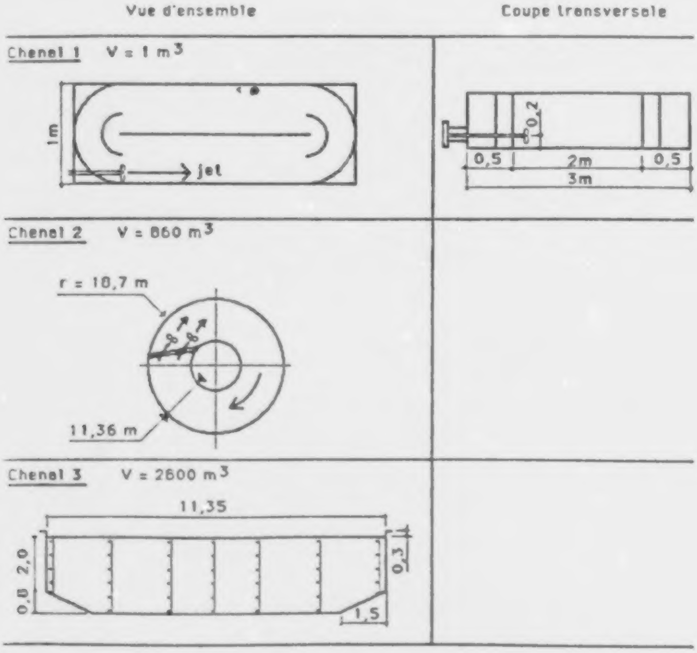
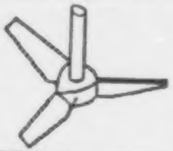
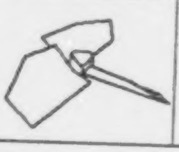


Tableau 1 : Caractéristiques géométriques des bassins

	Type de mobiles		diamètre d (m)	Np	Nqp
Chenal 1	Hélice marine		0,144	0,35	0,55
	Turbine 3 pâles a 310		0,150 et 0,198	0,35	0,68
Chenal 2 et 3	Turbine 3 pâles TMI		1,3	0,94	0,84

* Puissance dissipée $P = Np . f d^3 d^5$
Débit de pompage $Qp = Nqp N d^3$

Tableau 2 : Caractéristiques des mobiles d'agitation

RESULTATS EXPERIMENTAUX

1. Circulation

1.1. Chenal 1

Pour des raisons de facilité, une étude complète a été réalisée sur le chenal 1 (pilote) pour déterminer l'influence de la vitesse de rotation N et du diamètre du mobile d, sur la vitesse de circulation U. Pour un mobile donné, U est proportionnel à N (figure 1) :

$$\bar{U} = a N$$

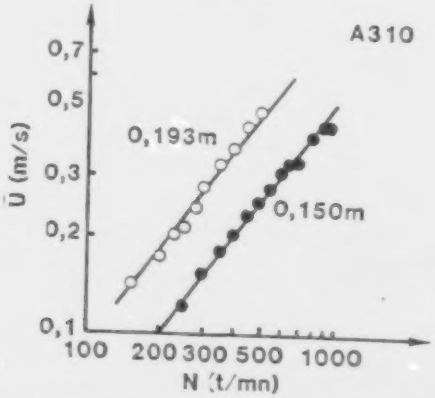


Figure 1 : Influence de la vitesse de rotation sur la vitesse de circulation pour deux diamètres du mobile (A310)

d = 0,150 m

d = 0,193 m

L'influence du diamètre du mobile se traduit par :

$$\bar{U} = \text{cte } N\, d^x$$

où $x = 2$

La puissance dissipée nécessaire pour obtenir une vitesse de circulation donnée dépend du type et du diamètre du mobile. Le tableau 3 donne à titre d'exemple, les valeurs des puissances dissipées pour chaque mobile testé.

	d m	N t/mn	Pw w	N t/mn	P w
H.M.	0,144	444	8,8	666	29,6
Turbine A310	0,150 0,193	400 220	7,9 2,7	600 330	26,6 16
		U = 0,20 m/s		U = 0,30 m/s	

Tableau 3 : Puissances dissipées pour chaque mobile resté

La puissance dissipée diminue fortement quand le diamètre du mobile augmente. Son évolution peut être prévue à partir des résultats précédents:

$$\bar{U} = \text{cte } N\, d^x$$

$$P = Np\ \rho\ N^3\, d^5$$

soit : $P = \text{cte } U^3 . d^{-1}$

Il faut également remarquer que la puissance P est proportionnelle au cube de la vitesse de circulation U. Passer de 0,2 m/s à 0,4 m/s implique une multiplication de la puissance par 8. Il faudra donc bien préciser au préalable la vitesse nécessaire au maintien en suspension des solides présents, pour ne pas dépenser inutilement de l'énergie.

Enfin, l'étude de similitude a montré que pour des mobiles d'agitation de type axial, la vitesse de circulation U était liée par la relation suivante :

$$\bar{U} = (4\ Nqp\ /\ \pi).\ N\ d\ (d\ /Deq)$$

ou Deq est le diamètre équivalent du chenal.

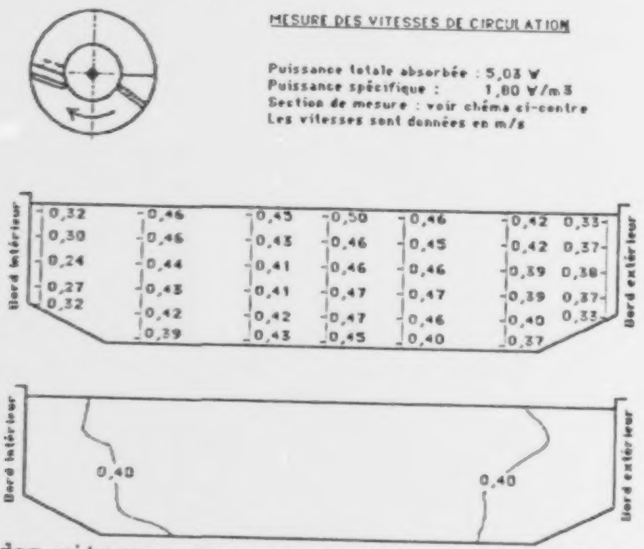
Autrement dit, pour obtenir dans deux systèmes géométriquement semblables (facteur d'échelle K) la même vitesse de circulation, il suffit de maintenir la vitesse périphérique $N\ \pi\ d = Cte$ aux deux tailles d'installation. La conséquence de ce critère d'extrapolation est que la puissance dissipée par unité de volume P/V (w/m³) est multipliée par le facteur K^{-1}. P/V diminue donc quand on passe d'une petite échelle à une grande échelle, tout en maintenant la vitesse de circulation identique. Ceci remet donc en cause le critère P/V = 30 w/m³.

1.2. Chenal 2

Le relevé du profil de vitesses a permis de tracer une carte de vitesse sur une section transversale du chenal. Un résultat type est montré sur la figure 2. La figure 3 donne l'évolution de U (m/s) en fonction de la vitesse de rotation N du mobile d'agitation :

$$\bar{U} = 0{,}45\ N$$

m/s t/s



MESURE DES VITESSES DE CIRCULATION

Puissance totale absorbée : 5,03 W
Puissance spécifique : 1,80 W/m3
Section de mesure : voir chéma ci-contre
Les vitesses sont données en m/s

Bord intérieur
0,32
0,30
0,24
0,27
0,32
0,46
0,46
0,44
0,43
0,42
0,39
0,45
0,43
0,41
0,41
0,42
0,43
0,50
0,46
0,46
0,47
0,47
0,45
0,46
0,45
0,46
0,47
0,46
0,40
0,42
0,42
0,39
0,39
0,40
0,37
0,33
0,37
0,38
0,37
0,33
Bord extérieur
Bord intérieur
0,40
0,40
Bord extérieur

Figure 2 : Carte des vitesses sur une section transversale (chenal 2)

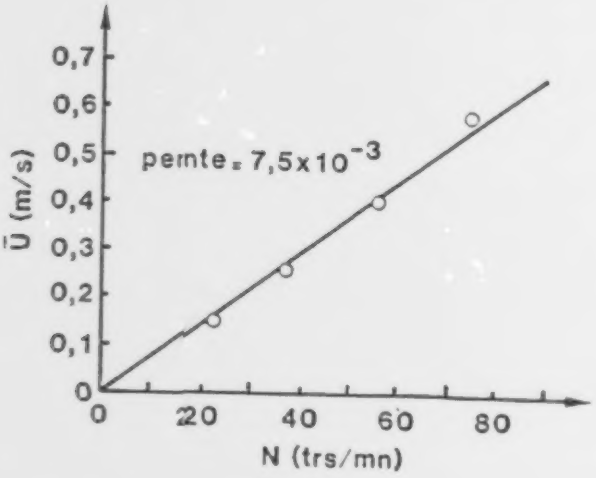


Figure 3 : Evolution de la vitesse de circulation en fonction de la vitesse de rotation N (chenal 2)

Si la relation établie au paragraphe précédent est utilisée, on trouve U = 0,44 N, ce qui est tout à fait en accord avec les résultats expérimentaux obtenus. Quant à la puissance dissipée P/V, elle varie entre 0,22 et 4,5 w/m³. Les résultats trouvés à l'échelle industrielle sont donc en accord avec ceux prévus à partir de l'étude laboratoire.

1.3. Chenal 3

Le relevé du profil de vitesses a permis de déterminer la vitesse moyenne U de circulation en fonction de la vitesse de rotation N des deux mobiles d'agitation. La figure 4 donne l'évolution de U en fonction de N.

$$\underset{m/s}{\bar{U}} = 0{,}50 \underset{t/s}{N}$$

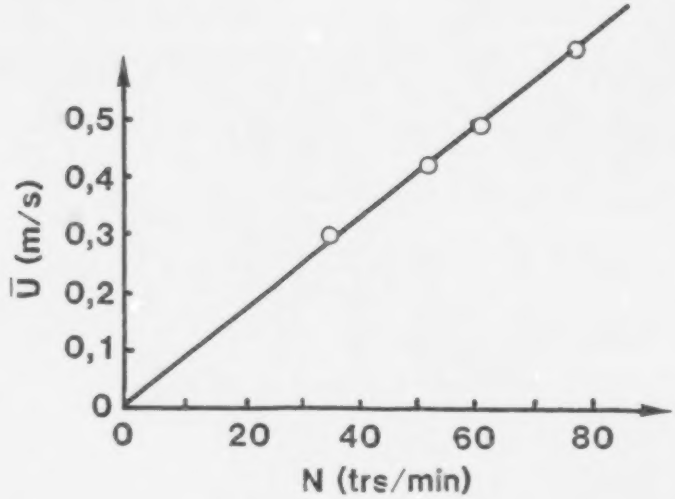


Figure 4 : Evolution de la vitesse de circulation en fonction de la vitesse de rotation N (chenal 3)

La relation établie au pragraphe 1.2. donnerait Ū = 0,57 N, ce qui est en accord avec les résultats trouvés.

Il faut préciser que deux relevés de profil de vitesse ont été faits sur deux sections du chenal diamétralement opposés. Les valeurs de Ū trouvées sont identiques.

2. Circulation et aération

L'étude de la circulation et de l'aération a été faite sur le chenal 3. L'aération est assurée par des diffuseurs DR2 Degrémont. Les 1100 diffuseurs sont répartis en 5 panneaux qui en comportent chacun 220 et placés à 0,15 m du fond du bassin. Chaque diffuseur est constitué d'une rondelle élastique percée d'un trou de 2,9 mm et fixée sur la tuyauterie (espacement 0,30 m). Chaque diffuseur (moyennes bulles) débite 1 Nm³/h d'air.

L'ensemble des résultats expérimentaux obtenus est reporté sur le tableau 4.

N (trs/mn)	0	74	61	52	35
P (kW) circulation	-	9,3	5,2	3,2	0,98
Ps (W/m^3) circulation	-	10,7	6	3,7	1,1
$\bar{U}$ (m/s)	-	0,62	0,49	0,42	0,29
Ps (W/m^3) circulation + aération	9,1	19,9	15,2	12,9	10,3
k_{La} (h^{-1})	1,62	2,35	2,21	2,02	1,52
Apport horaire (kg O_2/h)	14	20,7	10,3	17,8	13,3
Apport spécifique nominal (kg O_2/kWh)	1,75	1,2	1,47	1,80	1,50
$\bar{U}'$ (m/s) avec air	-	0,59	0,46	0,39	0,21

Tableau 4 : Influence de l'aération sur la circulation

* La contribution de la puissance dissipée au niveau de l'aération (9,1 w/m^3) est importante par rapport à la puissance totale dissipée.

* En présence d'air, la vitesse de circulation U diminue très peu par rapport à celle obtenue sans air. Les bulles d'air en montant ne créent pas un "barrage" à l'écoulement liquide horizontal.

* Les bulles en ascension libre seules permettent d'obtenir un apport horaire d'oxygène de 14 kg O_2/h. La circulation engendrée par les mobiles d'agitation a pour effet d'augmenter la quantité d'oxygène transférée. Un gain de 50 % est obtenu pour N = 74 tr/mn. Cependant, les valeurs de l'ASN, qui tiennent compte de l'énergie fournie pour introduire l'air et pour créer la circulation, sont légèrement inférieures à celle obtenue avec les bulles seules. Une valeur optimale est obtenue pour N = 52 tr/mn, correspondant à une vitesse moyenne de circulation de 0,39 m/s, ce qui est très correct pour empêcher les dépôts de solides.

CONCLUSIONS

L'étude faite, à la fois sur une installation laboratoire et sur deux installations industrielles, a permis de mettre au point une méthode permettant de prévoir la circulation engendrée dans des chenaux par des mobiles d'agitation à axes horizontaux. La méthode demanderait à être affinée en fonction d'autres géométries de chenaux.

L'introduction d'air sous forme de bulles, par des panneaux répartis au fond du chenal, permet d'assurer le transfert d'oxygène ; la circulation engendrée permettant d'augmenter le transfert d'oxygène de façon très significative. Cette solution permet de pouvoir réguler facilement l'apport d'oxygène dans le bassin tout en maintenant une circulation correcte avec une puissance dissipée faible.

Une étude complète est actuellement en cours sur le chenal 3.

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RÉSUMÉ DE LA CONFÉRENCE PROPOSÉE POUR LE 11IÈME SYMPOSIUM SUR LE TRAITEMENT DES EAUX USÉES

TITRE: Le premier filtre biologique au Québec et au Canada: une solution unique au problème particulier de traitement des eaux usées de la Municipalité du Canton d'Orford.

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Le petite municipalité du Canton Orford est située aux pieds d'un des plus grands et pittoresques centres de ski au Québec, soit le Mont Orford. Habituellement, la population de résidents permanents représente une fraction de la population de pointe pendant la saison de ski, créant ainsi un problème environnemental unique. Et quoi de plus pour le compliquer que le fait que le milieu récepteur, la rivière Aux Cerises, se décharge en amont de la place municipale d'un des plus beau lac de l'Estrie, soit le lac Memphrémagog.

L'énorme variation des débits et charges organiques, de saison en saison et de semaine en fin de semaine, a exigé une approche particulière lors de la conception de la station d'épuration.

Les délais associés à l'ouverture du centre de ski modernisé alourdissait la complexité mais, par contre, ont fourni un défi et une occasion de plus aux concepteurs pour démontrer leurs connaissances, leurs créativités et habilités de gestion de projets. Une collaboration exemplaire entre tous les intervenants au dossier (Ministère de l'Environnement, Ministère du Tourisme, la Municipalité et les consultants) a permis de concevoir, de construire et de mettre en opération la station d'épuration dans un délai exceptionnel de douze (12) mois.

....2

Pour traiter les importantes variations de débits et charges, une partie du décanteur Imhoff existant a été convertie en bassin d'égalisation. Un système de contrôle par microprocesseur limite les charges hydrauliques sur les unités de traitement subséquentes. La charge organique sur ces unités (variation 1:10) a été égalisé par un traitement primaire physico-chimique et polymères suivi d'une clarification dans un décanteur lamélaire. Cette étape de traitement permet également d'atteindre un très haut niveau de déphosphatation (0,3 mg/l).

Le traitement biologique est assuré par deux (2) cellules de filtres biologiques aérés de type BIOCARBONE. Cette filière de traitement n'exige pas une décantation secondaire ni un traitement tertiaire, donc l'effluent filtré est directement désinfecté aux rayons UV avant de rejoindre la rivière Aux Cerises.

Les objectifs de traitement visés (DBO_5 : 15 mg/l; MES: 15 mg/l; P_t: 0,5 mg/l et coliformes fécaux « 900 c.f./100 ml) sont largement atteints et même dépassés. Les premiers résultats démontrent en effet que l'effluent atteint de façon consistante et ce, en phase de mise en service, 10, 10 et 0,3 mg/l de DBO_5, MES et P respectivement.

CHARACTERIZATION OF SUSPENDED SOLIDS IN EFFLUENTS FROM BIOLOGICAL FILTERS

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ABSTRACT

Some of the physical properties of suspended solids in two trickling filter effluents were measured. Several size indices and shape factors of these particles were determined with the use of an image analysis system. The majority of the particles had diameters of less than 100 microns. The applicability of the sizing technique was limited to particles of which the longest dimension was greater than 10 microns.

The settling velocities and the sizes of the particles studied were measured in a quiescent water column by a multiexposure photographic technique. Results showed that particle settling velocity increased as a fractional power of size. Based on these direct measurements, the particle density was calculated, and found to have a linear relationship in a logarithmic scale with the particle size.

A comparison of the physical properties of filter influent-effluent solids confirmed that particles in filter effluents were of a different nature from those in the influents.

KEY WORDS

Trickling Filter Effluent, Suspended Solids, Particle Geometric Properties, Settling Velocity, Density/Porosity-Size Function.

INTRODUCTION

To achieve a high quality effluent from a biological wastewater treatment system, it is axiomatic that not only must the microbial species be effective in removing the desired pollutants, but the microorganisms themselves must also be effectively removed from the treated wastewater after they have fulfilled their metabolic role. In biological filtration, the generally inferior performance of the process as compared with activated sludge treatment, has been related to the unsatisfactory settling characteristics of solids that are washed out of the process (Bank et al., 1976; Forster, 1977; and Sarner, 1986). Consequently, the challenge of producing a final effluent which is reasonably free from suspended solids has been one of the most difficult problems in many high-rate biological filtration plants.

It is usually believed that effluents from trickling filters contain some residual wastewater solids and some biological solids. The latter are generated by periodic or continuous sloughing of the biological film developed on the filter medium. In the high-rate systems, these solids are highly putrescible, have a relatively high oxygen demand, and are light and flocculent (Babbitt and Baumann, 1958). However, little is known about their characteristic features such as settling velocity, size and size distribution, shape, density and the interrelations of these factors.

An understanding of the physical properties of the effluent particles may provide more insight into their behavior and nature, which may in turn lead to more effective approaches for their removal. The particle settling velocity, density and size distribution may determine the fraction of the solids that will appear in the final effluent and thus improve understanding of some of the factors influencing sedimentation of trickling filter effluent solids. Moreover, these properties may reflect the effectiveness of the biological filtration process and a more profound elucidation of the overall process may emerge.

The principal objective of this study was to quantitatively investigate some of the physical characteristics of particles in effluents from biological filters. This research is merely a stepping stone to further studies aimed at improving the clarification step that follows the biological filtration process.

MATERIALS AND METHODS

Trickling Filter Plant

The source of biological filter effluent samples for this study was the Harmony Creek Water Pollution Control Plant No. 1 in Oshawa, Ontario. The plant is a high-rate trickling filter plant and, on average, processes 27,000 m^3/day (6.0 M.G.D) of wastewater. Raw wastewater undergoes preliminary treatment (screening, grinding of the screenings, and grit removal) and then primary sedimentation. The partially-treated sewage flows by gravity into three high-rate trickling filters. The effluent from the filters flows into two secondary clarifiers from which a portion of the filter effluents is recirculated to the primary tanks. The secondary sludge is returned to the primary settling tanks. The effluent from the secondary clarifiers is chlorinated and then discharged into Lake Ontario. Table 1 summarizes the plant performance for the period of the study. The second filter was not included as its effluent was combined with that of filter No. 1 in the outlet box of filter No. 2.

The three trickling filters are of identical construction, each being 38 m (125 ft) in diameter and 1.75 m (5.7 ft) deep. Flow is applied to the stone media contained in each filter, using two rotating arm distributors. Based on the average daily flow of 27,000 m^3/day and the average primary effluent BOD concentration of 67 mg/L, the organic and hydraulic loadings on each filter are, respectively, 300 g. per cu.m. of media per day (19 lbs. per cu.ft. per day) and 8 cu.m. per sq.m. of surface area per day (7 M.G. per acre per day).

Measurement of Particle Geometric Properties

Grab samples of trickling filter effluent were taken from the filter outlet chamber, and the solids were kept suspended by a continuous, gentle agitation of the sample with a glass rod. About 10 mL of the obtained liquor was withdrawn with a wide-mouth pipette (5 mm in diameter) and poured into a petri-dish. The sample was then solidified by adding to the petri-dish 10 mL of a liquified, 1.5% agar solution which had been cooled to 50°C. Upon addition of the agar solution, solid particles were dispersed uniformly in the agar solution by gently rotating the dish. The solidified sample was then transferred to the laboratory where it was stored at a temperature of about 1°C, thereby preventing any alteration of the prepared sample until its analysis on the next day.

Two filters were considered in this part of the study from which three replicate samples were taken from each filter. The samples were taken at the same time and treated in the same way. The operating data of the studied filters on the day of the experiments are presented in Table 2.

The particle characteristics were determined by employing the image analysis system, OMNICON 3000 (Li and Ganczarczyk, 1986). In this research, two objective lenses, 8x and 16x, were used in order to cover a wide range of

TABLE1 -- PLANT PERFORMANCE FOR THE PERIOD, NOVEMBER 20, 1986 TO MARCH 20, 1987.

Influent*	Range**	Mean**
Flow(m3/day)	---	27000
pH	5.4-7.0	---
BOD	147-186	168
COD	326-365	344
SS	130-176	151
Primary effluent		
BOD	70-150	130
COD	200-276	229
SS	56-83	72
T.F. 1 effluent		
BOD	50-90	70
COD	---	130
SS	45-80	70
T.F. 3 effluent		
BOD	38-70	54
COD	---	136
SS	40-70	60
Secondary effluent		
BOD	30-55	40
SS	20-45	30

* Samples were taken after the grit removal tanks.

** All the reported values are in mg/L except of pH and flow data.

particle sizes. Particles were selected and detected by using a variable frame, controlled by joysticks on the monitor keyboard, which could change the size and position of the frame and move it anywhere on the screen. Measurements were performed on particles from many fields, selected at random, with an average of about 300 particles from each sample.

TABLE 2 -- OPERATIONAL CHARACTERISTICS OF THE STUDIED FILTERS DURING THE EXPERIMENTS

Parameters	Trickling Filter - 1	Trickling filter- 3
Organic Loading	480 g/m^3/day (30 lb/1000 ft^3/day)	480 g/m^3/day (30 lb/1000 ft^3/day)
Hydraulic Loading	8 m^3/m^2/day (7 M.G./acre/day)	8 m^3/m^2/day (7 M.G./acre/day)
BOD Infl./Effl.	130/55 mg/L	130/65 mg/L
COD Infl./Effl.	200/115 mg/L	200/125 mg/L
SS Infl./Effl.	79/ 55 mg/L	79/60 mg/L
pH Effl.	6.0	5.9
Temp. Effl.	18^{o}C	19^{o}C

To examine whether the agar solution, by which the tested samples were prepared, was free from particulate matter (e.g. dust, undissolved agar particles, air bubbles, etc.), two control samples (1.5% agar solution only) were also tested by the image analyzer.

Settling Velocity Experiment

The settling velocities of individual particles settling in a quiescent water column were measured, utilizing the multi-exposure photographic technique developed by Li and Ganczarczyk (1987), modified by the addition of a small plastic tube for sample introduction (Fig. 1) to let the particles settle through the focused area.

Suspended solids samples were taken directly from a filter effluent and the particulate matter was kept in suspension by gently agitating the liquid with a glass rod. The volumes of 1.0 mL of the solids suspension were withdrawn by a wide-tip opening pipette (5 mm in diameter) and carefully deposited into the quiescent tap water column through the plastic tube which was immersed at least 5 cm below the surface of the water. As the samples which were introduced had very low solids concentrations owing to the nature of the liquid leaving the biological filter, the provision of the plastic tube was indispensable in limiting particles dispersion to the focused region. The chance for more particles to go through the focused area was, thereby, substantially increased.

Settling distance and particle size measurements were made from the resultant photographic negatives, by the image analyzer, BIOQUANT II (Li and Ganczarczyk, 1988).

Dry Density of Particle

A specific gravity bottle was employed for dry density measurement. Because of the relatively low concentration of suspended solids in the plant secondary sludge, a large volume sample of about 200 litres was allowed to

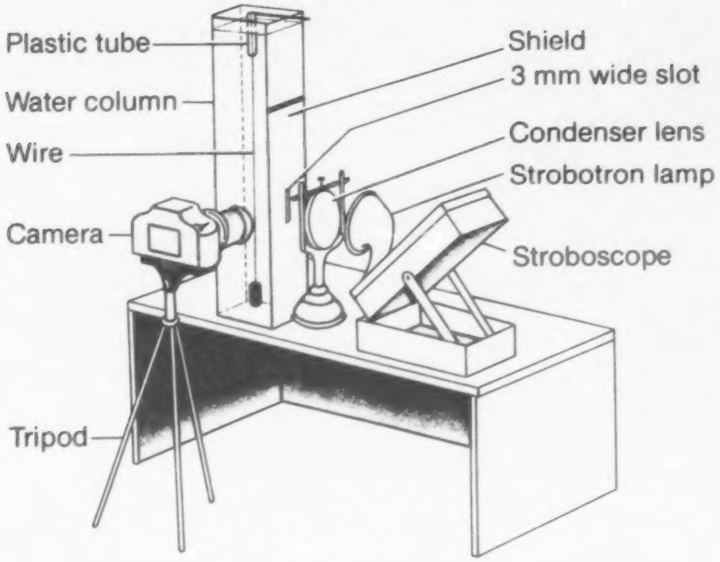

Fig. 1 --- Experimental apparatus for particle settling velocity measurement

settle for 1 hour, and the sludge obtained was collected in plastic jars and transferred to the laboratory. To effect rapid sedimentation of fine particles, the sludge was then centrifuged in 100-mL centrifuge tubes for 15 minutes at 5000 rpm. The concentrated cakes were dried in an oven at 105°C for 24 hours, and the dry solids were ground in a mortar.

Using a funnel, a solid sample (5% to 10% of the bottle volume so as to reduce weighing measurement errors) was transferred to a pre-weighed specific gravity bottle. The bottle, containing the particle sample, along with the stopper, was weighed and then half-filled with distilled water. Without inserting the stopper, the bottle was placed in a bell jar and subjected to a vacuum in order to remove the adsorbed or trapped gases among the fine particles. Care was taken not to let the water and solids mixture boil so as to prevent solids loss. With the same caution, water addition and vacuuming were repeated until the bottle was filled to the tip of the neck.

The filled bottle was then immersed, up to its neck, in a constant temperature water bath for 15 to 20 minutes to allow the bottle and its contents to reach the bath temperature (22°C in this study). After the stopper was carefully placed into the bottle to avoid any entrapment of air bubbles, the bottle was lifted from the bath by the neck and the water residue wiped off with a tissue. The bottle was reweighed with its contents.

Eventually, the weight of the same gravity bottle used for the preceding measurement, containing distilled water only, was determined by following the above procedures again. The following equation was used for the calculation of dry density:

$$P_S = \frac{P_W}{1.0 - (WS / DS)} \qquad \ldots\ldots\ldots\ldots\ldots(1)$$

where P_s is the dry density of the particle (g/cm^3), P_w is the density of distilled water, WS is the difference in weight between the specific gravity bottle containing water plus particles and the bottle containing only water (mg), and DS is the difference in weight between the bottle containing the dry solids and the empty bottle (mg).

RESULTS AND DISCUSSION

Geometric Characteristics of Particles

Based on the preliminary evaluation of the agar solution used for sample preparation, particles less than 10 microns in the longest dimension were not considered and were excluded from this study as the control samples contained particles as big as 9.5 microns in the longest dimension.

Histogram graphs, showing typical distributions of some size indices and shape factors for two filter effluents, are shown in Figs. 2-7. Since all the geometric parameters (other than the shape factors) covered several orders of magnitude, equal widths of intervals could not be maintained. Thus, the selected scales are arbitrary in order to reveal the detailed distribution over the entire parameter range.

A study of typical distributions of particle longest dimension for filters 1 and 3 effluents (Fig. 2) shows that most of the particles in filter 1 effluent, about 26%, fall in the size range of 25-50 microns, whereas the majority of the particles in filter 3 effluent, around 28%, have their longest dimension in the range of 100-200 microns. Nevertheless, more than two thirds of the particles from either effluent are in the size range of 25-200 microns. The same observation is valid for the distributions of equivalent circular diameter (Fig. 3) which is defined as the diameter of a circle having the same projected area as the particle cross-sectional image.

In view of the fact that trickling filter effluent particles may contain an entire spectrum of shapes, characterizing the particles by more than one independent parameter would be mandatory in order to have a better definition of particle shape. Fig. 4 shows typical breadth distributions of suspended solids in filters 1 and 3 effluents.

The distributions of the above geometric parameters appear close to log-normal. As a typical example, the log-normal probability plot of particles breadth is depicted in Fig. 5. It is apparent that the plotted points are skewed to the right and that the lower tail is shorter than would be expected if the data came from a log-normal distribution. Moreover, if a line were drawn to follow the plotted points, a bimodal distribution could be detected on the log-probability paper as reflected by a change in the slope of the line.

Considering a two-dimensional image, two shape factors were investigated to describe quantitatively the deviation of the particle configuration from that of a sphere. The elongation ratio (ratio of the longest dimension to the breadth) distributions (Fig. 6) showed that the vast majority of the particles, about 80%, possessed elongation ratios of less than 2.0, which suggested that the longest dimensions of most particles did not deviate from breadths by more than a factor of two. The circularity (ratio of the area with the longest dimension, as the particle diameter, to the measured projected area of a particle) distributions as presented in Fig. 7 showed that about three quarters of the particles had circularity values less than or equal to 2.2, confirming the previous conclusion that most particles were relatively regular in shape.

Table 3 presents a summary of the mean values of particle-geometric characteristic distributions for the two filter effluent samples. No

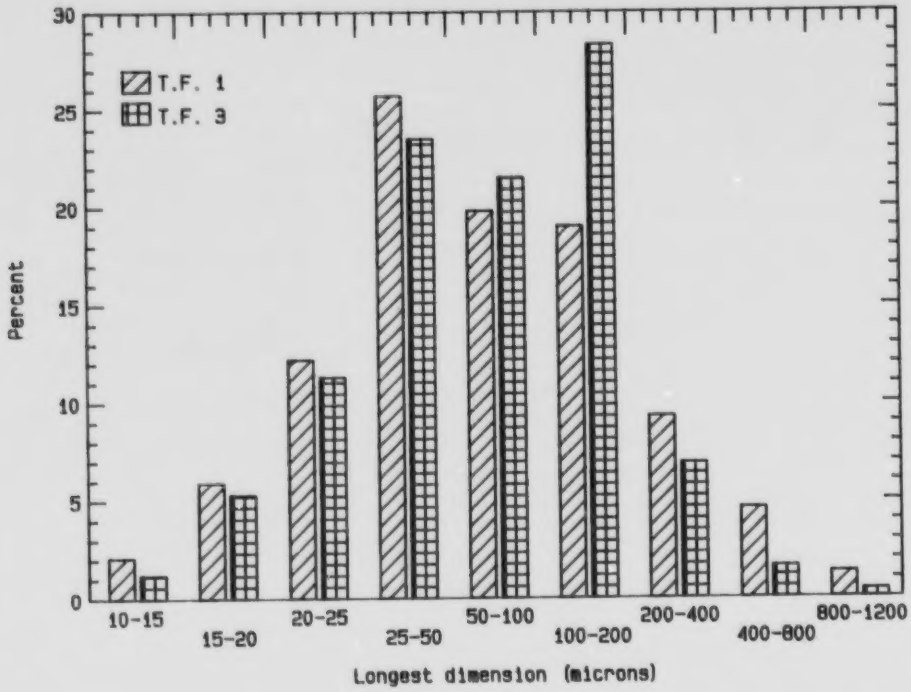

Fig. 2 --- Distribution of suspended solids longest dimension

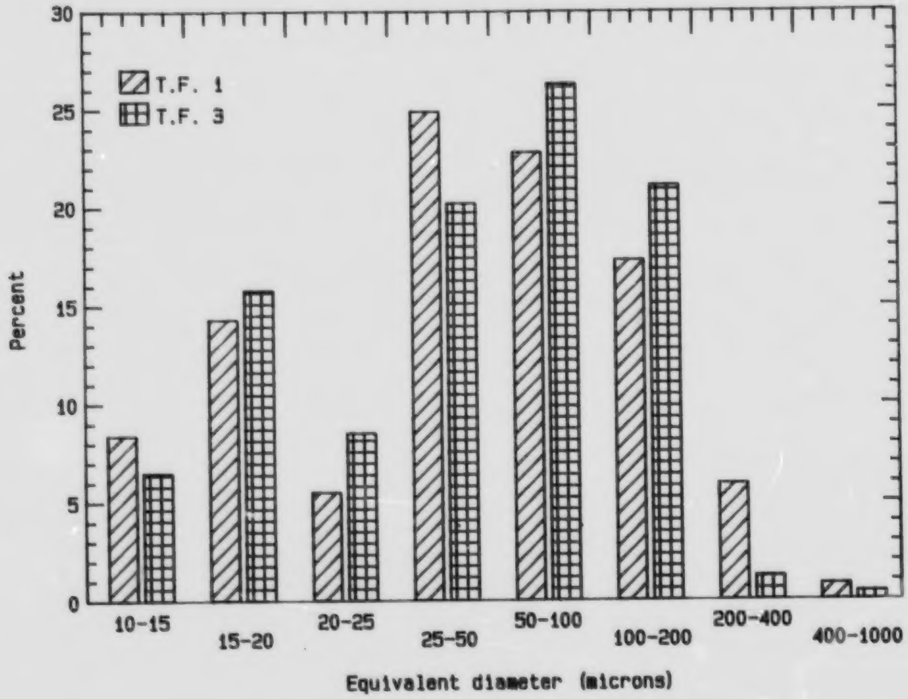

Fig. 3 --- Distribution of suspended solids equivalent diameter

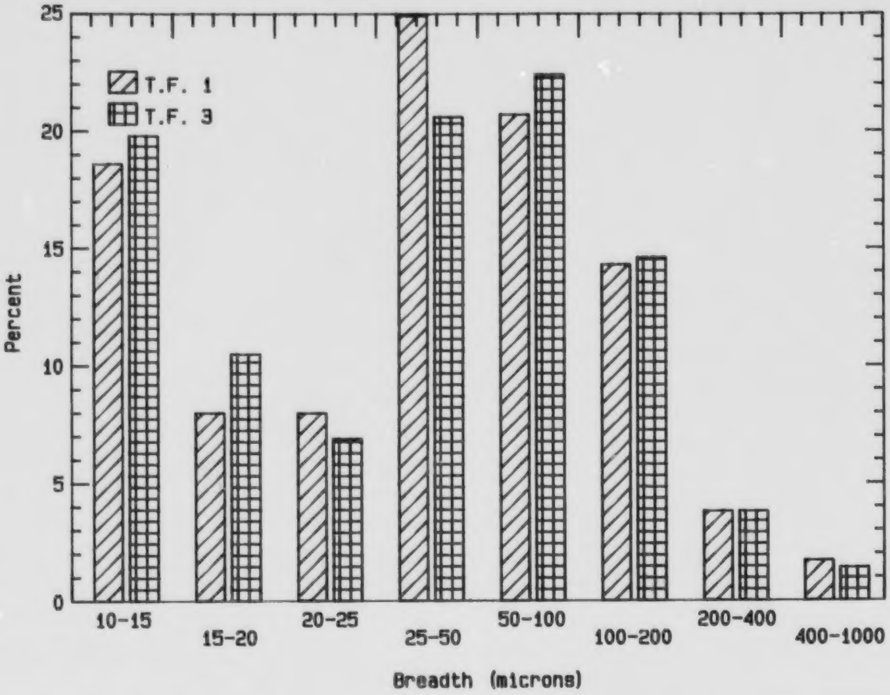

Fig. 4 --- Distribution of suspended solids breadth

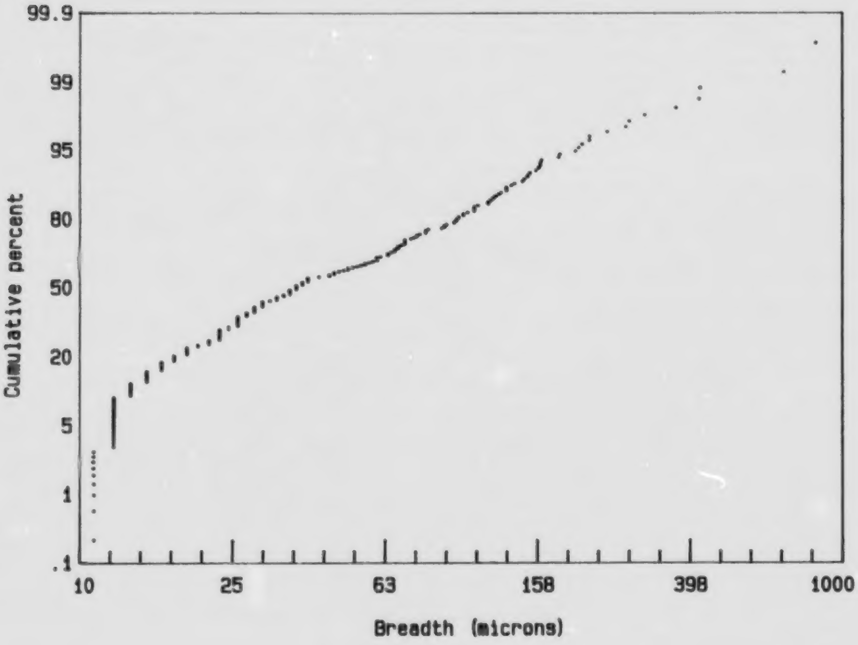

Fig. 5 --- Log-normal probability plot for suspended solids breadth distribution

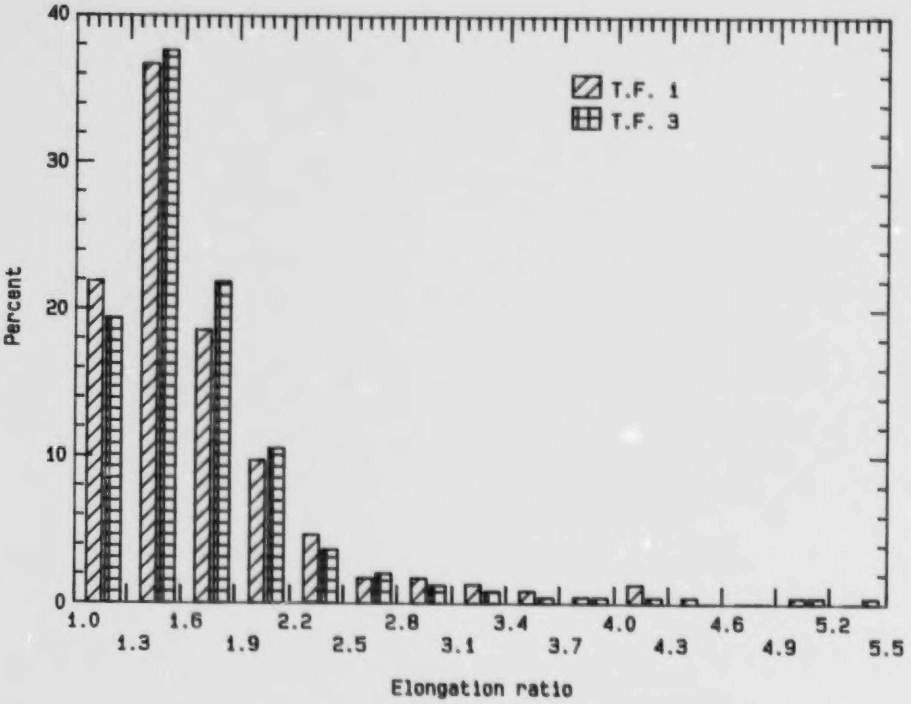

Fig. 6 —— Distribution of suspended solids elongation ratio

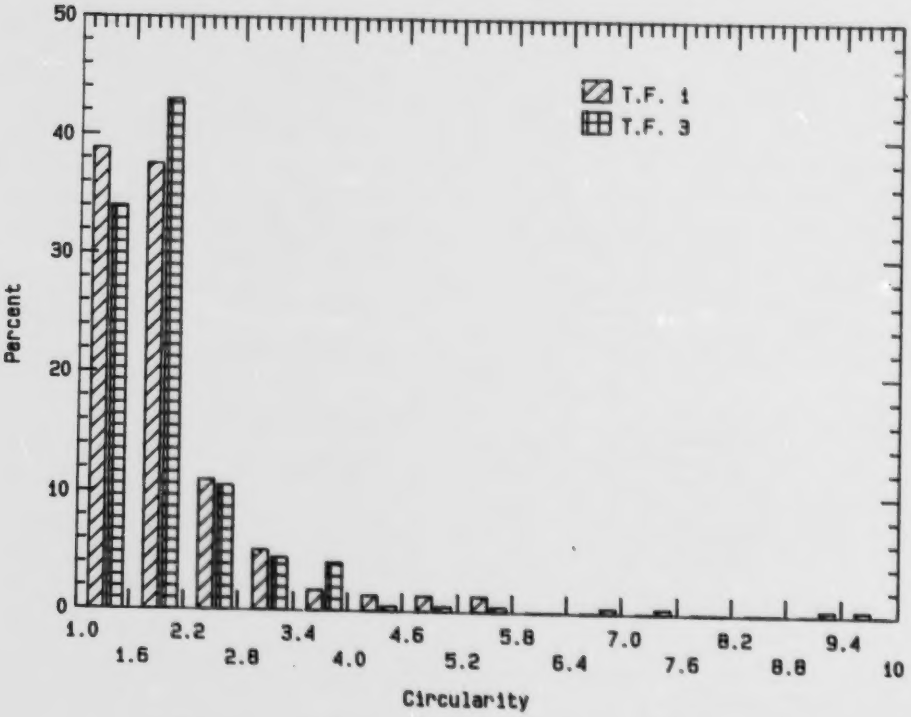

Fig. 7 —— Distribution of suspended solids circularity

TABLE 3 -- MEAN VALUES OF PARTICLE-GEOMETRIC CHARACTERISTICS DISTRIBUTIONS, AND STATISTICAL RESULTS OF OVERALL MEANS COMPARISON.

Geometric Parameter		Longest dimension (microns)	Breadth (microns)	Projected area (micron^2)	Perimeter (microns)	Elongation ratio (---)	Circularity (---)	Equivalent diameter (microns)
Trickling filter No. 1 samples	1	93.3	57	5130	305	1.7	2.35	62
	2	122.3	72.2	15872	416	1.8	2.40	78.7
	3	116.9	68	12229	399	1.72	2.25	77.2
Overall mean		110.8	65.7	11077	373	1.74	2.33	72.2
Trickling filter No. 3 samples	1	97.6	51.3	7147	317	1.83	2.59	58.2
	2	108.2	62.6	9922	385	1.77	2.55	66.7
	3	95	58	5828	353	1.71	2.19	64.7
Overall mean		100.4	57.3	7632	352	1.77	2.44	63.2
L.S.L. (%)		12	7	8	64	5.5	15	8

L.S.L: The Lowest Significant Level at which the two overall means are significant.

significant difference between the two overall means of any parameter was evident as reflected by the lowest significance level, at which the two means would be different, which was higher than 0.05.

Particle Settling Velocities

As shown in Fig. 8, the settling velocities of sixty five particles were plotted against their average diameters (averge values of longest and shortest dimensions). In this experiment, almost all the particles were in the Stokes' law range (i.e Reynolds' numbers < 1.9; Rich, 1961) during their settling. Yet unlike Stokes' law, particle settling velocity increased as a fractional power of the characteristic dimension; in other words, the measured settling velocities had a size dependence lower than that predicted by Stokes' law.

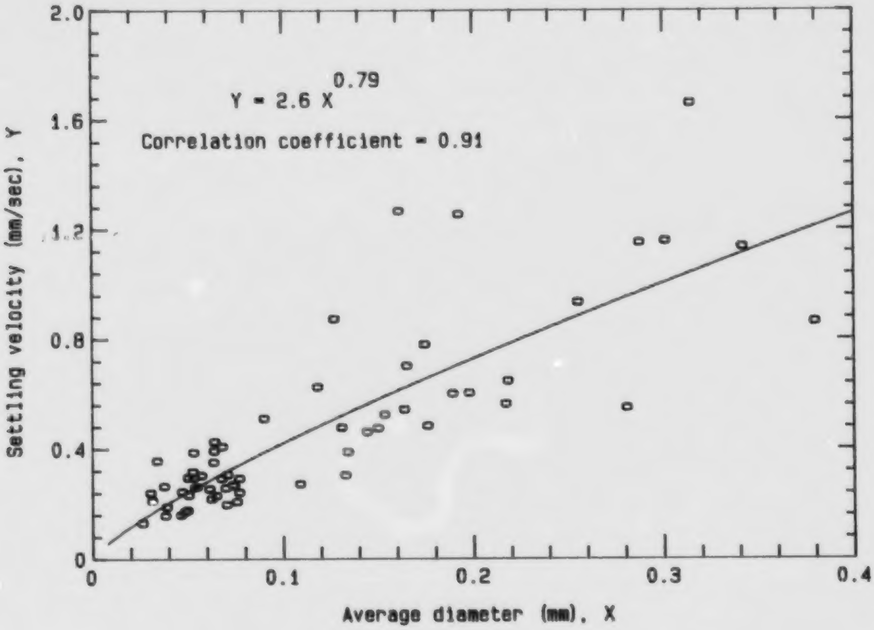

Fig. 8 --- Settling velocity as a function of average diameter

It has long been known that the particle shape, settling orientation, and variation of particle density with size have some influence on the settling velocity-size relationship. Recently, Li and Ganczarczyk (1988) have experimentally confirmed the hypothesis of fluid flow through biological particle aggregates undergoing gravitational settling and, thereby, presented another factor that had a substantial effect on aggregate settling velocity. Consequently, the traditional computation of settling velocity by Stokes' law from the size and density measurements, without considering the effect of these parameters, may seriously bias the results.

Trickling filter effluent generally is characterized by a low solids concentration of a relatively flocculent character. Because of this feature the settling behavior of a particle is not influenced, to any noticeable extent, by the presence of other particles. Therefore, particle settling velocity measurement by the multi-exposure technique described here can determine the settling properties of the particles which make up the suspension leaving biological filters.

The Size-Density/Porosity Relationships

As a particle aggregate was assumed to consist of two parts, a solid part and a void (water) part, the particle porosity (e) could be expressed as (Tambo and Hozumi, 1976; and Tambo and Watanabe, 1979):

$$e = 1.0 - \frac{P_p - P_w}{P_s - P_w} \qquad \ldots\ldots\ldots\ldots\ldots\ldots(2)$$

in which the density of the solid material in the particle was assumed to be the particle dry density (P_s). Based on the direct measurements of the particle settling velocity and the particle size, the particle wet density (P_p) was calculated using Stokes' law of free settling. As indicated earlier, therewere a number of uncertainties involved in the density calculation from Stokes' law. The introduction of these factors into Stokes' law, however, would be at the expense of greater mathematical complexity. Therefore, the use of Stokes' law for density calculation was regarded as an appropriate approximation.

The calculated densities expressed as buoyant densities (particle density minus that of water) and the corresponding particle diameters were found to have a straight line relationship on logarithmic scales as illustrated in Fig. 9. Thus the relationship can be expressed as follows:

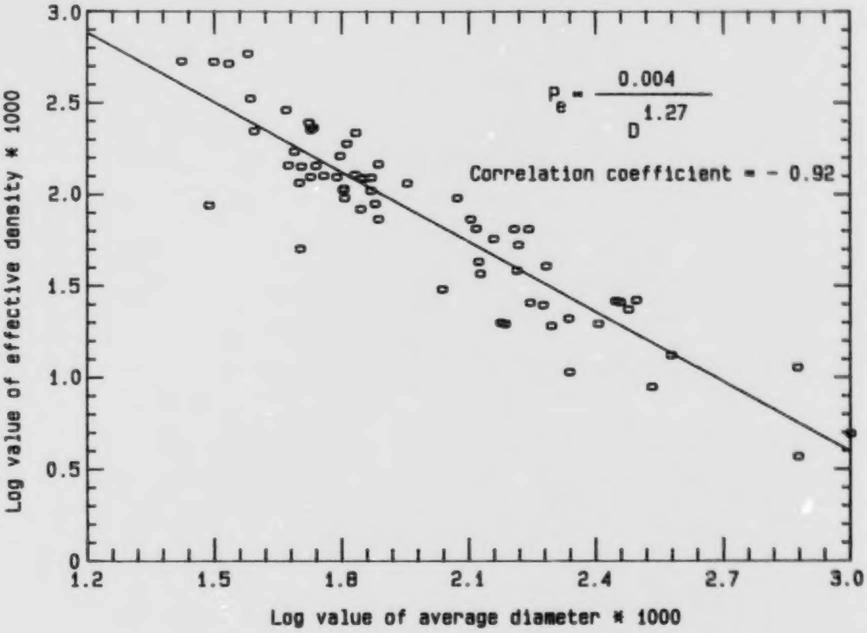

Fig. 9 --- Effective density (P_e) as a function of average diameter (D)

$$P_e = P_p - P_w = \frac{0.004}{D^{1.27}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots(3)$$

where P_e is the particle effective density or buoyant density (g/cm^3) and D is the particle average diameter (mm). The two constants 1.27 and 0.004 represent the slope of the straight line and the effective density of a 1.0-mm diameter particle, respectively. The former parameter reflects the rate of change of density with size. Lagvankar and Gemmell (1968) have reported a floc size-density relationship for iron (III) flocs while Tambo and Watanabe (1979) have reported the same relationship for aluminium, iron, magnesium, and activated sludge flocs. Generally, all the reports concerning the size-density relationship for microbial or organic aggregates showed a decrease in particle density as size increased.

The mean dry density of four replicate samples was found to be 1.69 g/cm^3 with a standard deviation of ± 0.028 g/cm^3. The presence of inert solids in trickling filter effluent can be linked to the high value of dry density. Accordingly, by substituting the measured dry density and the calculated size-density function (eq. 3) into the porosity equation (eq. 2), the size-porosity function can be expressed as:

$$e = 1.0 - \frac{0.0058}{D^{1.27}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots(4)$$

where e is the particle porosity (%). Fig. 10 shows this function. It can be seen that a small increase in the average diameter of particles less than 100 microns results in a drastic increase in porosity. Particles larger than 100 microns demonstrate a slower increase in the rate of porosity. This finding is consistent with that of Li and Ganczarczyk (1987). Their experimental results showed that the porosity of activated sludge flocs increased at two different rates with the increase of the floc longest dimension. The transition point occurred at a size around 200 microns.

The porosity of a particle aggregate is contributed to by the water in the interstitial spaces within the particle aggregate. The microbial components of a particle contain a substantial amount of water which should not be considered or included as a part of the liquid between the fine particulates of which a particle is composed. The dry density, however, does not exclude the water in the microbial cells. Furthermore, an aggregate can be composed of particulate matter (organic and/or inorganic) that has a wide range of densities. Consequently, the porosity function may overestimate particle porosity.

The variation of particle density and/or porosity with size is of significant importance with regard to the settling properties of suspensions in effluents from biological filters. Usually, particles of more than 100-micron diameters (settleable solids) are effectively removed by typical sedimentation operations (Faisst, 1980). According to this study, such particles had densities approaching that of water but sufficient to develop enough velocity to settle. This can probably be explained by the fluid flow through porous particles which has been proved to have a significant effect on the particle settling velocity. Theoretically, a solid sphere, having the same size and density as a highly porous sphere, will settle slower than the porous sphere as the internal permeation within the high porosity-permeable sphere reduces the hydrodynamic resistance that will be experienced by the solid

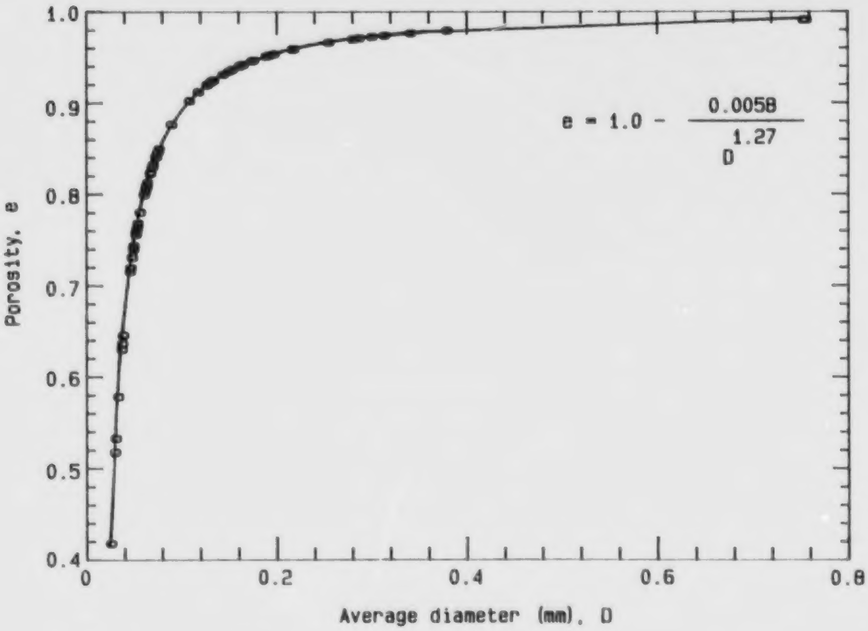

Fig. 10 --- Variation of particle porosity with average diameter

sphere (Neale et al., 1973). According to Li and Ganczarczyk (1988), the presence of a coke, carbon, or resin carrier within an activated sludge floc has impeded the flow through the floc and, therefore, the carrier floc settled at rates no higher than that of a comparable size floc formed without a carrier, in spite of the fact that the carrier was denser than the control floc. In this study, a 100-micron particle is predicted by equation 3 and 4 to have, respectively, a density of 1.07 g/cm^3 and a porosity of 0.89. Thus, it is possible that the fluid flow through the porous particles has caused them to develop enough velocity to settle, while having little density difference with respect to water.

Nonetheless, particle size distributions of trickling filter effluent showed that a large portion of the particles was in the size ranges of less than 100 microns. Also, the measured settling velocities of these particles were so small that the turbidity of the final effluent would be attributed to them. Thus, a requirement for a flocculation of these particles prior to settling appears obvious.

Filter Influent-Effluent Solids Characteristics

With the aid of the multi-exposure photographic technique, along with the image analyzer, BIOQUANT II, physical properties of influent-effluent solids from a trickling filter were determined and compared. The influent sample was taken directly from the rotating arm distributors so as to obtain a representative sample. An effluent sample was taken and examined on the same day.

A detailed comparison of influent-effluent particles settling velocity distributions is given in Fig. 11 in which the settling velocities are averaged over each size range. As depicted, the difference in settling velocity was not substantial for particles less than 280 microns in size. The majority of influent particles, about 92%, were over this size range. At the other extreme of size, the settling velocities of influent particles were much higher than those of effluent particles. However, there was a negligible number of influent particles in that size range as compared to effluent particles. Yet the experimental data demonstrated that the settling velocity for influent and effluent particles was in the range of 167 to 7750 µm/sec, and 120 to 2530 µm/sec, respectively. Apparently, the presence of sewage solids in the primary effluent influenced the settling rate of the particles entering the filter as these solids were denser than the solids leaving the filter, and thus settled faster.

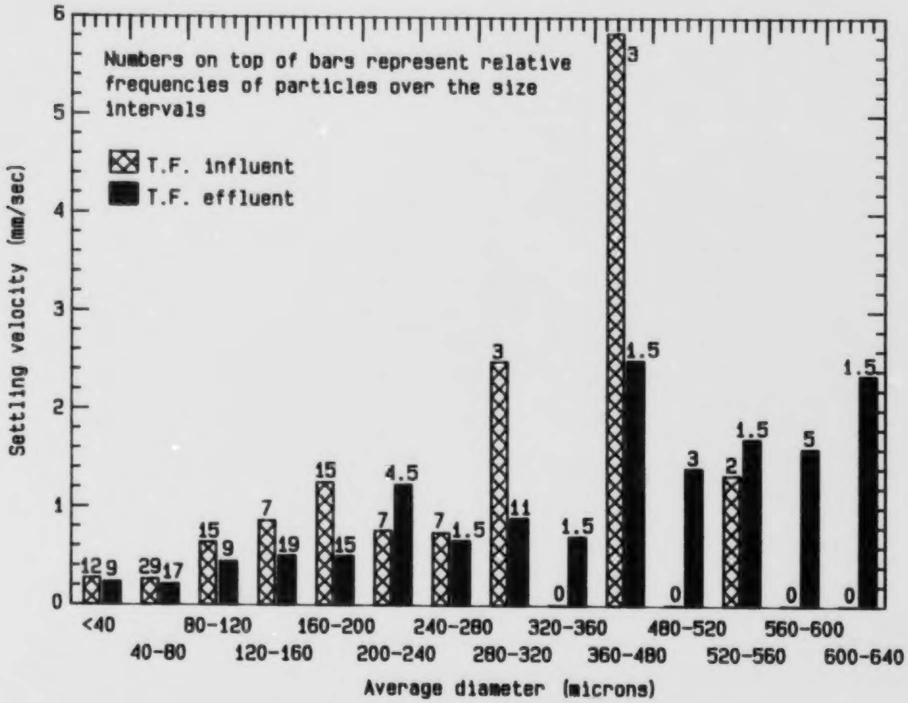

Fig. 11 --- Comparison of influent-effluent particle settling velocity distributions as function of size distributions

Indeed, the calculated density distributions presented in Fig. 12 confirmed this hypothesis and demonstrated that a large fraction of effluent particles, around 75%, possessed a density of less than 1.08 g/cm^3; whereas, only about half of the influent particles had their densities in that range.

Clearly, particles in filter effluent were both larger in size and had a lower density than those in the influent.

CONCLUSIONS

1. The larger than 10-microns particle size measurements for the suspended solids in the effluents from two identical high-rate trickling filters, operated under the same hydraulic and organic loadings, showed that the particles generally ranged to 1375 microns in length, to 912 microns in width, and to 943 microns in equivalent diameter, with overall means of 106,

62, and 68 microns, respectively.

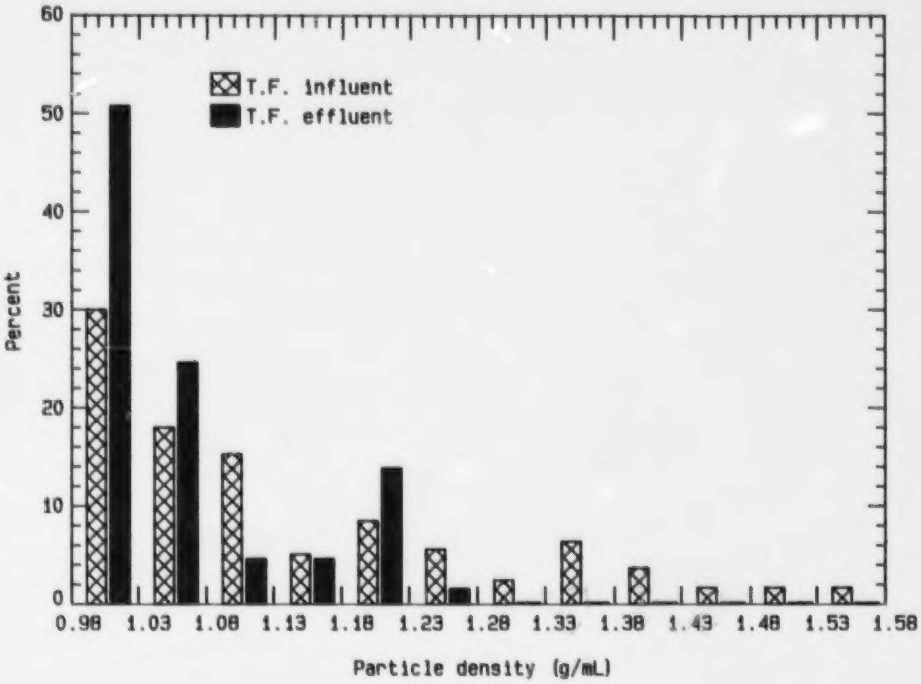

Fig. 12 --- Comparison of influent-effluent particle density distributions

2. On average, about 80 percent of the measured particles had their equivalent diameters in the range to 100 microns.

3. The settling velocities of particles in the studied filter effluents increased as a fractional power of the particle size. The average particle settling velocity was in the range of 0.66 ± 0.08 mm/sec.

4. A correlation between size and density was recognized; the density decreased (porosity increased) with increasing particle size.

5. The density-decreasing rate (porosity-increasing rate) for particles smaller than 100 microns in size was very much higher than that for particles larger than 100 microns.

6. The physical properties of filter influent-effluent solids lead to an expected conclusion that the particles in the filter influent are effectively captured in the treatment system, and those released in the effluent are of a biological nature.

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Influence de la contribution relative idustrie-municipalité sur la biocinétique réactionnelle d'étangs aérés.

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Le programme d'assainissement du Ministère de l'Environnement du Québec étant bien amorcé, de nombreuses municipalités du Québec doivent s'attaquer à la conception d'un système de traitement de leurs eaux usées. Dans bien des cas des industries sont comprisent dans les limites de la municipalité et un traitement conjoint de ces effluents (municipal et industriel) doit être envisagé. Dans ces conditions il en résulte un effluent de qualité très variable d'une municipalité à l'autre. Afin de réaliser la conception du système d'épuration sur des bases rationnelles, il faut connaître les paramètres de design.

Lorsque le système d'épuration projeté est constitué d'étangs aérés facultatifs, c'est le paramètre de design K_e, du modèle cinétique d'ordre 1 suivant:

$$r_s = K_e S_e$$

qui doit être connue. L'équation de design

$$\frac{S_e}{S_o} = \frac{1}{1 + K_e t_h}$$

dont le terme de droite est généralement corrigé par un facteur de correction "F.C.", est dérivée de l'équation précédente.

Comme la constant K_e est dépendante de la température, l'utilisation de l'équation d'Arrhénius modifiée permet d'évaluer la valeur de K_e pour toute température comprise dans la plage d'expérimentation. L'équation permettant d'évaluer K_e à une température T étant:

$$K_{eT} = K_{e20} \; \theta^{(T - 20)}$$

La valeur de la constate K_e est dépendante de la nature du substrat polluant. Ainsi dépendamment de la contribution relative et de la qualité de l'effluent industriel mélangé à l'effluent urbain, la valeur de K_e pourra être très différente. La relation possible entre K_e et la charge massique de DCO industrielle dans le mélange sera donc examinée.

Le présent travail évalue l'effet de la température sur la constante K_e du modèle d'ordre un $r_s = K_e S_e$ pour un effluent mixte donnée et modélise l'influence de la contribution industrielle sur K_e.

CONSIDERATIONS THEORIQUES

Modèles biocinétiques

C'est le modèle de Monod (1950) qui est à la base du dévelopement du modèle d'Eckenfelder (1968).

Selon Monod:

$$\frac{r_s}{X} = \frac{k\,S_e}{K_s + S_e} \qquad (1)$$

où r_s/X est le taux d'enlèvement du polluant par unité de masse de biomasse
où k est une constante spécifique fonction de la température et du polluant particulier
où S_e est la concentration résiduelle en polluant
où K_s est une constante dite de taux moitié

Lorsque la concentration S_e est faible, le modèle d'ordre 1 est satisfaisant; on a alors :

Ordre 1:

$$\frac{r_s}{X} = K_1\,S_e \qquad (2)$$

où K_1 est la constante spécifique d'assimilation du polluant de ce modèle.

Pour une concentration en biomasse, X, à peu près constante, Eckenfelder propose d'utiliser :

$$r_s = K_e\,S_e \qquad (3)$$

où K_e, la constante spécifique du modèle, est le taux spécifique d'enlèvement du polluant (d^{-1}); à nouveau, sa valeur dépend de la température d'opération et de la nature du polluant.

La façon de mesurer r_s, pour un système continu, est de faire:

$$(S_o - S_e/\ t_h) \qquad (4)$$

où S_o est la concentration initiale en polluant et
où t_h est le temps de résidence hydraulique

Les étangs aérés facultatifs sont caractérisés par le modèle Eckenfelder. On peut donc, de ce qui précède, obtenir:

$$\frac{S_o - S_e}{t_h} = K_e\,S_e \qquad (5) \implies$$

$$\frac{S_e}{S_o} = \frac{1}{1 + K_e\,t_h} \qquad (6)$$

Un facteur de correction (F.C.) est généralement introduit dans le terme de droite de l'équation (6) pour compenser l'influence des dépôts de boues dans le système.

A une température donnée, une valeur plus grande de K_e montre un polluant plus facilement biotraitable.

Conséquemment un plus grand K_e, pour une même efficacité désirée de biotraitement, permet des temps de résidence, t_h, plus faibles pour un même volume de bioréacteur.

Pour un même polluant, une diminution de température entraînera une diminution de la valeur de K_e. En effet l'influence de la température peut être prédite par l'équation d'Arrhénius modifiée:

$$K_{eT} = K_{e20} \, \theta^{(T - 20)} \qquad (7)$$

où: 0 est le " coefficient de température "; l'équation prévoit que la valeur de K_e décroît si la température diminue.

L'équation (7) prend la forme linéaire suivante:

$$\ln K_{eT} = (\ln \theta) * T + cte \qquad (8)$$

Evaluation de K_e à une température T constante

La façon de mesurer r_s de l'équation (3) pour un système en cuvé, comme c'est le cas pour une caractérisation biocinétique par suivi de la DBO, est de faire:

$$r_s = - (ds/dt) \qquad (9)$$

qui remplacée dans l'équation (3) donne:

$$- (dS/dt) = K_e S_e \qquad (10)$$

Si la DBO est utilisée pour mesurer S_e, (10) devient:

$$- (dL/dt) = K_e L \qquad (11)$$

où L est la DBO restante au temps t.
La forme intégrée de (11) devient:

$$L = L_o \, e^{-Ke\ t} \qquad (12)$$

où t est le temps pour atteindre L
où L_o est la DBO restante au temps 0, c'est à dire la DBO ultilme.

Comme le paramètre de suivi est plutôt la DBO exercée, Y, on introduit :

$$Y = L_o - L \qquad (13)$$

et l'équation (12) devient:

$$Y = L_o (1- e^{-Ke\ t}) \qquad (14)$$

Un suivi continu à température constante, de la DBO exercée (Y) en fonction du temps (t) requis, permet d'évaluer K_e lorsque la valeur de L_o est connue, ce qui n'est pas le cas. Plusieurs méthodes existent pour contourner ce problème dont la méthode des Moindres carrés, Metcalf & Eddy (1979).

Courbe de la DBO

Pour déterminer la constante K_e de l'équation (3) on réalise un suivi de la DBO, qui consiste à mesurer la DBO exercée par un ensemencement de micro-organismes, à intervalles réguliers. En portant sur graphique la DBO exercée en fonction du temps on obtient une courbe asymptotique comme celle de la figure 1.

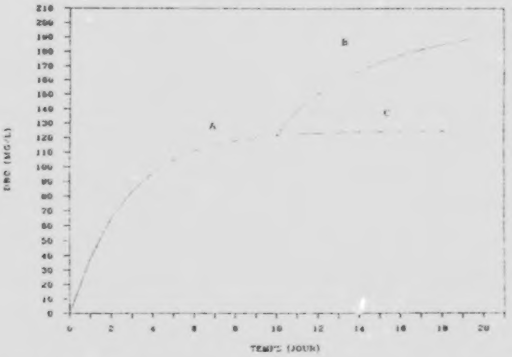


figure 1: COURBE TYPIQUE DE LA DBO EXERCEE EN FONCTION DU TEMPS

La portion A de la courbe, dite "de première phase", dure entre 6 et 10 jours et c'est ensuite la seconde phase qui débute, portion B de la courbe. Cette seconde phase est attribuable à l'action des bactéries nitrifiantes. Ces dernières ayant un taux de reproduction faible, on ne note pas leur action avant une période de 6 à 10 jours. On obtient la courbe C s'il n'y a pas prolifération des bactéries nitrifiantes. Dans le cas d'étangs aérés facultatifs, le design est basé sur l'enlèvement de la DBO carbonnée, représentée par la courbe C. En ajoutant un produit chimique, il est possible d'inhiber l'action des bactéries nitrifiantes et on obtient ainsi la courbe C. Young J.C. (1973) montre que le TCMP (2-chloro-6-(trichlorométhyle) pyridine) est le plus efficace afin d'inhiber les bactéries nitrifiantes sans inhiber les autres type de bactéries tout en gardant ses propriétés durant une longue période.

A la courbe de la figure 1 est associés deux paramètres. Le premier qui renseigne sur la biodégradabilité du substrat est K_e, la constante d'assimilation du polluant (d^{-1}). Le second renseigne sur la concentration totale en substrat polluant et s'appelle DBO ultime (L_o mg/l). La variation de K_e est présentée à la figure 2, pour une même valeur de L_o et une même température.

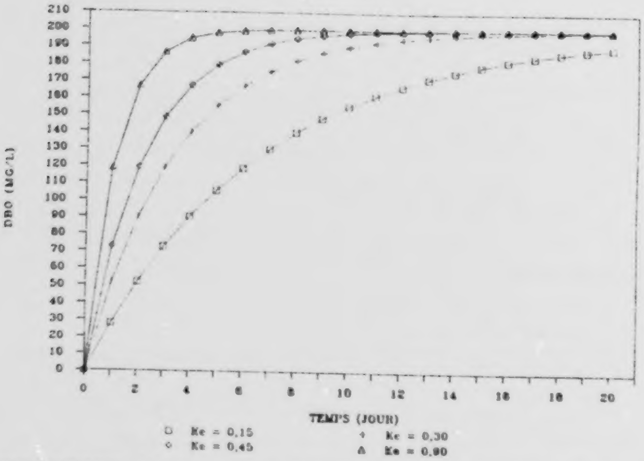


figure 2 : VARIATION DE K_e POUR DIFFERENTS SUBSTRATS POLLUANTS

Pour une même DBO ultime, un substrat qui est facilement biodégradable possède une valeur de K_e élevée par rapport à un substrat moins biodégradable. La DBO ultime est la concentration totale de matière organique biodégradable, après un temps très long. En pratique on considère avoir atteint la DBO ultime après 20 jours. C'est généralement le cas d'un effluent municipal. Dans le cas de certains effluents industriels cela peut être plus court ou plus long.

Dans certain cas, lorsque l'effluent dont on mesure la DBO ne contient pas de micro-organismes, on doit en ajouter. Ces micro-organismes doivent être adaptés au substrat sans quoi on aura une phase de latence comme celle de la figure 3. On rencontre ce phénomène dans plusieurs cas, soit un effluent contenant des métaux lourds, qui sont toxique pour les micro-organismes, soit dans le cas où l'effluent a été chloré ou que l'on effectue le suivi à base température. Dans tout ces cas on doit acclimater des micro-organismes au substrat afin de les ensemenser dans les bouteilles à DBO.

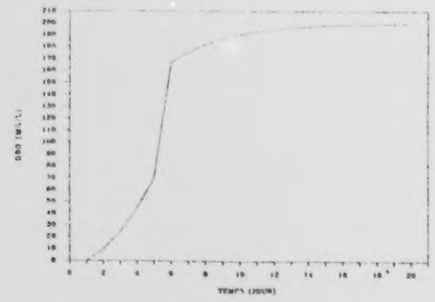


figure 3 : EFFET D'ACCLIMATATION DES MICRO-ORGARNIMES SUR LA DBO

Matériels et méthodes

On distingue deux phases concernant la procédure expérimentale, soit une phase d'acclimatation et une phase d'expérimentation en vue de la caractérisation biocinétique. Chacune de ces phases a une durée de 20 jours. La phase d'acclimatation a pour but de développer des micro-organismes (culture mixte) adaptés au substrat et aux conditions de température auxquels ils seront soumis lors de leurs ensemencements dans les bouteilles de DBO. La phase d'expérimentation consiste à suivre l'évolution de la DBO dans le temps afin d'obtenir la courbe de la DBO exercée, requis pour évaluer le taux d'enlèvement de la DBO (K_e).

L'effet de la température sur la constante K_e sera évalué en effectuant simultanément quatre suivis à quatre températures différentes (2 °C, 8 °C, 14 °C et 20 °C) pour un effluent mixte donné. En effet le type de micro-organismes varie en fonction de la température tout comme leur taux spécifique de croissance. On distingue trois types de micro-orgamismes soit les psychrophiles, les mésophiles et les thermophiles, chacun se distinguant par des plages optimales de température différente. On trouve à la figure 4 l'effet de la température sur la croissance des différents types de micro-organismes. Le taux spécifique d'enlèvement du polluant K_e varie de la même façon que le taux spécifique de croissance μ, en fonction de la température.

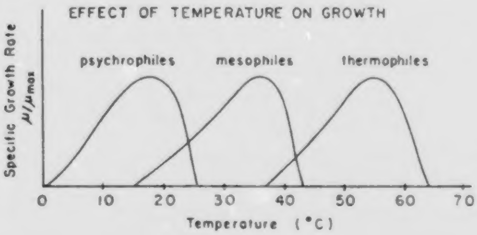


figure 4 : VARIATION DU TAUX SPECIFIQUE DE CROISSANCE EN FONCTION DE LA TEMPERATURE.

Le présent travail est effectué en deux volets. Le premier consiste à déterminer les paramètres de design K_e et θ pour un effluent mixte donné. Le second volet vise à déterminer l'effet de l'augmentation de la charge organique industrelle sur la valeur de K_e. La procédure expérimentale sera divisée en deux parties afin de décrire chacun des volets.

1° volet: Détermination de K_e et de θ.

Acclimatation

Lors de l'ensemencement de micro-organismes dans une bouteille à DBO, il faut que ces derniers soit habitués aux conditions expérimentales, soit la température de même que le substrat et sa concentration. La quantité de micro-organismes ensemencée est faible par rapport à la quantité de substrat qui l'entoure. L'acclimatation des bactéries à ces conditions est faite dans quatre réacteurs biologiques séquentiels (RBS) dont le temps de résidence hydraulique est quatre jours et l'âge des boues est 20 jours. Un temps de résidence hydraulique de 4 jours permet d'habituer les micro-organismes à une forte concentration en substrat alors que 20 jours d'âge de boues permet d'avoir une bonne variétée de micro-organismes.

L'acclimatation des micro-organismes se fait dans quatre réacteurs de 1,5 litres. Ils sont maintenus à 2 °C, 8 °C, 14 °C et 20 °C. Initialement le liquide mixte provenant d'une station d'épuration municipale est utilisé. L'aération du réacteur se fait à l'aide d'air saturé en humidité ce qui minimise l'évaporation et permet d'assurer le mélange du liquide mixte. Quotidiennement, 0,075 litre de liquide mixte est prélevé afin de maintenir 20 jours d'âge de boue. Suite à ce prélèvement l'aération est coupée, la décantation débute et dure une heure, à la suite de quoi 0,300 litre de surnageant est siphonné. L'aération est repartie et 0,375 litre de substrat est ajouté au RBS.

La température des réacteurs biologiques est controlée par un incubateur, pour les réacteurs à 2 °C, 8 °C et 14 °C et par un bain thermostaté pour le réacteur à 20°C. Afin de s'assurer que la température désirée est constante, une sonde thermique, relié à un thermomètre situé à l'extérieur de l'incubateur, est introduit dans chacun des réacteurs et la lecture du thermomètre est effectuée à plusieurs reprises au cours de la journée. Durant les 20 jours d'acclimatation la température est maintenu constante à ± 1 °C.

Le substrat polluant utilisé est celui qu'a livré l'équipe technique du MENVIQ et qui provient de la municipalité de St- Félix- de- Valois. L'échantillonnage d'une durée de 24 heures est proportionnel au débit.

Sur réception, l'eau brute est entreposée à 4 °C. Chaque jour un volume est prélevé, aprés s'être assuré de son homogénéité. Cette quantité est amenée à la température du RBS (2, 8, 14 et 20 °C), afin de minimiser l'effet thermique du substrat sur la population de micro-organismes. L'alimentation des RBS se fait à partir de ce substrat acclimaté thermiquement. Cette procédure à été suivie durant les 20 jours de l'acclimatation.

2° volet: Effet de l'augmentation de la charge organique industrielle sur K_e

L'acclimatation a nécessitée un bioréacteur (RBS) pour chacun des cinq substrats polluants, ces derniers étant composés d'une contribution croissante d'eau industrielle à l'eau urbaine. La façon de composer ces échantillons est exposée plus loin. La proportion d'eau industrielle dans le mélange, sur une base massique organique, étant de 0%, 25%, 50%, 75% et 100%. Les cinq substrats ainsi obtenus furent entreposés à 4° C durant toute l'acclimatation. Les conditions d'opération des RBS sont les mêmes pour les cinq réacteurs et sont identiques à celles exposées au premier volet. La température est la même pour tout les réacteurs RBS soit: 20° C ± 1° C.

Constitution des échantillons

La constitution des différents échantillons est effectuée sur la base du contenu organique, mesuré comme DCO. Ce choix découle du fait que c'est une mesure que l'on obtient rapidement (moins de 3 heures) et qu'il est important de composer les échantillons le plus rapidement possible après leur réception. On désire constituer des échantillons dont 25 %, 50% et 75% de la masse en DCO soit d'origine industrielle et dont le reste du pourcentage de la DCO soit dû à l'effluent urbain. Le pourcentage massique étant défini par:

$$\boxed{\%\ \text{Massique A} = \frac{\text{Masse du composé A}}{\text{Masse totale}}} \qquad (15)$$

devient, en terme de DCO:

$$\boxed{\%\ DCOi = \frac{Vi * Si}{Vi * Si + Vu * Su}} \qquad (16)$$

où:

% DCOi = % de la masse de la DCO industrielle dans le mélange
Vi et Vu = Volume d'eau usée industrielle et urbaine respectivement (l)
Si et Su = Concentration de matière organique de l'effluent industriel et urbain respectivement (mg/l)

On fixe le % DCOi de l'équation (16) et on mesure Si et Su; on doit donc déterminer les volumes nécessaires afin d'obtenir le mélange désiré. Pour ce faire on considère l'équation suivante:

$$Vi + Vu = Vt = 1 \qquad (17)$$

Un volume totale unitaire de 1 litre permet d'effectuer la composition du volume de substrat désiré. On obtient deux équations (16) & (17) et deux inconnus Vi & Vu. On résout pour déterminer les volumes nécessaires pour faire les compositions. La DCO résultante du mélange peut être calculée à l'aide de l'équation suivante:

$$DCOr = \frac{Vi * Si + Vu * Su}{Vi + Vu} \qquad (18)$$

où: DCOr = la DCO résultant du mélange.

Afin de s'assurer, lors du début des tests de DBO, que les mélanges sont bien ceux que l'on désire, la DCO des mélanges est mesurée. Les résultats sont comparés aux valeurs "théoriques" et présenté au tableau 1.

Tableau 1

COMPARAISON ENTRE LA DCO CALCULEE ET LA DCO MESUREE

% DCOi	DCO calculée (mg/l)	DCO mesurée (mg/ l)
0 %	222	222
25 %	274	251
50 %	366	367
75 %	539	518
100 %	1 025	1 025

Les échantillons ainsi constitués sont représentatifs des pourcentages massiques en substrat des eaux industrielles dans le mélange. La méthode proposée de constitution des échantillons est simple et facile à vérifier.

Suivi expérimental

La phase expérimentale consiste à suivre l'évolution de la DBO dans le temps. Ce suivi s'effectue au moyen de DBOmètres manométriques fabriqués par la compagnie Hach. Ces appareils sont placés dans des incubateurs maintenus à température constante. L'échantillon d'eau est placé dans la bouteille à DBO sans aucune dilution. Des éléments nutritifs sont ajoutés afin de palier à une éventuelle carence de ces derniers. Un inhibiteur de bactéries nitrifiantes est ajouté afin de mesurer uniquement la DBO carbonnée. Dans chaque bouteille est ajouté un volume de 10% de micro-organismes acclimatés au substrat et à la température d'intérêt. L'ensemencement est constitué du surnageant du RBS (décantation statique durant une heure). Des essais préliminaires ont permis de constater que c'est ce type d'ensemencement qui répond le mieux au critère du manufacturier et du Standard Methods (1975), à savoir que la correction apportée pour l'ensemencement doit être de 0,6 mg/l ou plus. Les lectures de la DBO sont effectuées quotidiennement à heure fixe durant une période de 20 jours.

Résultats et Discussions

Les résultats sont présentés en deux volets soit un volet pour chacune des parties de l'expérimentation.

1° volet: **Détermination de K_e et de θ.**

La valeur à 8 °C a été estimée à l'aide le la valeur de θ (un bris d'équipement n'a pas permis d'effectuer normalement le suivi à 8 °C). Les valeurs rapportées sont celles obtenues par la méthode des Moindres carrés (Metcalf & Eddy 1979) et K_e est exprimé dans la base népérienne (base e) à partir de mesures de la DBO et n'inclue pas la nitrification. Les valeurs pour l'échantillon de substrat mixte sont comparées aux valeurs suggérées par le MENVIQ (1982), pour un effluent urbain, par le biais de ses notes techniques (tableau 2).

Tableau 2

Température °C ± 1	St- Félix K_e d^{-1}	notes techniques K_e d^{-1}
2	0,27	0,11
8	0,37	0,16
14	0,58	0,25
20	0,53	0,37
θ	1,04	1,07

L'importance d'une caractérisation biocinétique est évidente à l'examen des valeurs du tableau 2. Ainsi l'équation de design (6) montre que pour une concentration d'alimentation et un débit fixe, le volume des étangs dépend uniquement de la valeur de K_e lorsque la valeur de la concentration de l'effluent est fixée. Dans le cas de St-Félix-de-Valois des étangs conçu avec les valeurs moyennes pour un effluent urbain auraient été trop grand. Basé sur une conception à basse température (2 °C) le volume aurait été plus de deux fois le volume nécessaire alors que basé sur une conception à 20 °C le volume des étangs auraient été 40 % trop grand: l'incidence sur les coûts de construction et d'opération (récurents) est importante.

2° volet: **Effet de l'augmentation de la charge organique industrielle sur K_e**

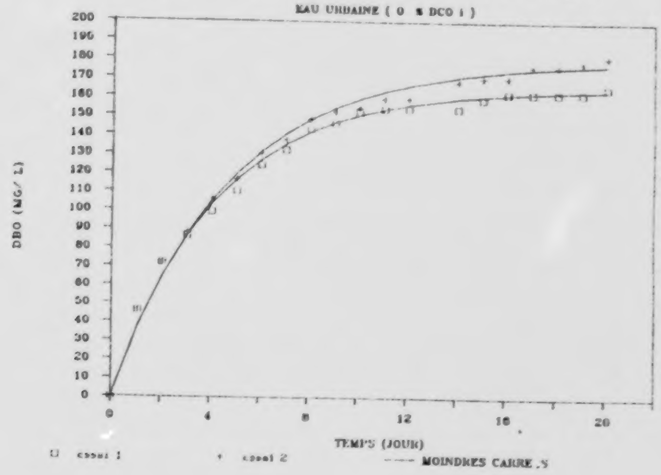
Les cinq suivis réalisés à 5 concentrations différentes d'effluent industriel mélangé à l'effluent urbain ont permis d'obtenir des valeurs représentatives de K_e. Les courbes de la DBO obtenues ne présentent pas la deuxième phase attribuée aux bactéries nitrifiantes; l'inhibiteur ajouté est donc efficace. Les valeurs de K_e et L_o obtenues sont représentatives de la phase carbonnée. Ces valeurs sont présentées au tableau 3. Le graphique 1 présente les résultats expérimentaux pour un des essais soit celui pour l'eau urbaine (0 % DCOi).

Tableau 3
CONSTANTES BIOCINETIQUES

% DCOi	K_e (d^{-1})	L_o (mg/ l)
0 %	0,23	172
25 %	0,26	184
50 %	0,40	264
75 %	0,47	402
100 %	0,53	727

Il est difficile de comparer les constantes biocinétiques obtenues lors du 2° volet et celles obtenues au 1° volet. En effet, dans le 2° volet l'effluent industriel n'a pas subi de traitement physico-chimique comme ce fut le cas pour l'effluent ayant servi pour le 1° volet.

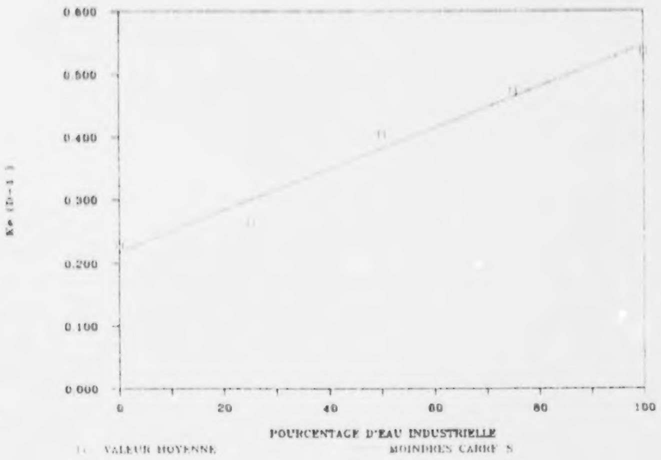
GRAPHIQUE 1: DBO EN FONCTION DU TEMPS



Dans le but de déterminer s'il existe une relation entre K_e et le pourcentage d'eau industrielle, un graphique des valeurs de K_e en fonction du pourcentage massique de la DCO industrielle a été tracé. Le graphique 2 présente les valeurs moyennes de K_e en fonction du % DCOi. Une relation linéaire permet de relier K_e au pourcentage de DCO industrielle dans le mélange utilisé. Le coefficient de régression linéaire (r^2) est de 0,97, ce qui indique qu'une droite est satisfaisante pour décrire le phénomène expérimental sous étude. L'équation de cette dernière étant:

$$\boxed{K_{e\ (\%)} = 0,22 + 3,29 * 10^{-3} * \%\ \text{DCO i}}$$

GRAPHIQUE 2: Ke EN FONCTION DU % DCO i



Pour chaque concentration étudiée, quatre essais ont été réalisés simultanément: deux essais dans les conditions optimum de mesure de la DBO et deux essais avec un facteur de sécurité de mesure de la DBO. Les valeurs de K_e obtenues pour les quatre essais ont été modélisées et la relation linéaire est toujours la meilleur méthode d'ajustement des points expérimentaux mais les deux "suivis de sécurité" n'ont pas été considérés dans le traitement des données donnant lieu aux résultats présentés dans le tableau no 3.

Conclusion

Le suivi fait avec l'échantillon frais de substrat, suite à l'acclimatation de micro-organismes durant 20 jours, ne montre aucune période de latence et présente uniquement la première phase de la DBO.

L'équation dérivée du modèle cinétique d'Eckenfelder, utilisée en design d'étangs aérés facultatifs, sera donc, pour l'échantillon d'effluent mixte frais:

$$\frac{S_e}{S_o} = \frac{1}{1 + 0,53\ t_h} * \text{F.C.} \qquad \text{à } 20\ ^\circ\text{C}$$

L'équation :

$$K_T = 0,53 * 1,04^{(T - 20)}$$

permettra de calculer K_e à toute température T

Par ailleurs il est possible de prédire la valeur de la constante bio-cinétique K_e, pour un mélange d'effluent urbain et industriel, à l'aide d'une équation linéaire. L'équation trouvée pour le mélange des eaux usées d'un abattoir de volailles et celles de la municipalité de St-Félix-de-Valois a la forme suivante:

$$\boxed{K_{e\,(\%)} = 0,22 + 3,29 * 10^{-3} * \%\ \mathrm{DCO}}$$

Compte tenue de cette relation linéaire il est possible d'estimer les paramètres de cette droite à partir de deux points, par exemple la valeur de K_e de l'effluent industriel et K_e de l'effluent urbain.

La pente de la droite dépend donc des effluents à mélanger, la valeur des constantes K_e des effluents " purs" déterminant la valeur de la pente. Le cas d'un effluent urbain moyennement à fortement chargé peut donner une valeur de K_e qui pourrait être supérieure à celle d'un effluent industriel difficilement biodégradable contenant, par exemple, des phénols ou une liqueur de pâte et papier. Dans ces conditions une pente négative montrerait l'influence "néfaste" de la contribution industrielle à l'effluent mixte.

L'avantage d'une telle relation est de pouvoir prédire la biodégradabilité de l'effluent mixte lorsqu'il y a variation de la contribution industrielle à la charge organique totale. Ainsi on peut déterminer la valeur de K_e que l'on aura dans quelques années en se basant sur les prévisions de production à long terme d'une industrie. Il est alors facile pour le concepteur du système de traitement de prévoir son efficacité à long terme. Cela lui permet de doter le système d'épuration de la flexibilité nécessaire afin de pouvoir se conformer dans le futur aux normes du Ministère de l'Environnement.

D'autres caractérisation avec d'autres types d'effluents devraient être réalisés afin de pouvoir confirmer la validité de la relation trouvée. Si cette relation reste valide, il demeure facile de prévoir la biodégradabilité d'un effluent mixte en connaissant la valeur de la constante K_e des effluents qui seront mélangés. Cela permettrait de prévoir l'effet de l'implantation d'une industrie dans une municipalité ayant déjà un système de traitement de ses eaux usées.

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START-UP OF A HIGH RATE ANAEROBIC SLUDGE DIGESTER

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A high rate anaerobic sludge digestion system, consisting of eight first stage covered digesters each of 4000 m^3 capacity followed by four secondary open digesters, is constructed for biological stabilization of sludges from 110,000 m^3/d sewage treatment facility at Tripoli, Libya. The digesters are designed to treat combined primary and waste biological sludge from activated sludge process using Simon-Hartley sludge digestion systems.

The first unit of the battery of primary digesters was commissioned with stored primary sludge in absence of proper seed sludge. An intermittent feed of primary sludge was adopted during initial stages. The feed solids were maintained around 8 percent in the primary sludge. In spite of operational care to maintain pH and temperature at optimum levels, the acid levels and volatile acid to alkalinity ratio of the digester contents remained significantly high. The gas production for the initial start-up period was highly variable, with at times negligible production. The methane percentage in the digester gas was mostly below satisfactory levels.

This paper presents discussion on the analysis of the start-up problems of the digester in light of the detailed data on quality of feed sludge, digester sludge, digester gas, feed pattern and digester conditions. The corrective measures adopted to stabilize the digestion process and their resulting effects on the performance of the digester are also described.

TRAITEMENT TERTIAIRE D'EAUX USÉES MUNICIPALES PAR CULTURE DE *SCENEDESMUS* SP EN INSTALLATION PILOTE

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Introduction

Le rejet de substances polluantes dans le milieu est devenu un motif croissant de préoccupation, compte tenu de l'impact écologique considérable que peut présenter le déversement quantitativement important d'effluents. À cette dimension s'ajoute la perte de substances potentiellement utiles pour la production de biomasse.

Dès 1946, Caldwell a proposé de considérer les substances polluantes des effluents urbains (NH_4^+, PO_4^{-3}) comme de véritables substances nutritives utilisables par les micro-algues d'eau douce. On sait que les micro-algues constituent un mode efficace de conversion photosynthétique de l'énergie solaire en biomasse, comme on peut le constater dans divers écosystèmes [Shelef & Soeder, 1980a]. L'humain a d'ailleurs exploité depuis longtemps le phénomène d'eutrophisation plus ou moins contrôlé pour accroître la productivité de certains systèmes aquicoles, tout spécialement en pisciculture [Bardach *et al*, 1972; Billard,1980 a, b].

Compte tenu de leur potentiel, les systèmes de traitement des eaux usées recourant aux micro-algues ont donc été assez rapidement jugés attrayants et un nombre croissant de travaux ont alors été effectués un peu partout dans le monde depuis les travaux des pionniers [*cf* Burlew, 1953]. On se doit ici de mentionner tout spécialement les travaux de Oswald et de ses collaborateurs [Gotaas & Oswald, 1951; Oswald & Gotaas, 1957; Oswald *et al*, 1957; Oswald, 1960] sur les étangs d'oxydation destinés à l'épuration des effluents domestiques et industriels, travaux qui ont mené plus tard Oswald et ses collaborateurs à développer des systèmes plus intensifs [*cf* Oswald, 1963].

Depuis, de nombreux travaux ont été menés sur le traitement des eaux usées par différents groupes de recherche, tout spécialement aux États-Unis, en Allemagne, en Israël, en Tchécoslovaquie et au Japon, trop nombreux en fait pour qu'on puisse en dresser une liste exhaustive. On trouvera dans Shelef et Soeder (1980b) la plupart des références importantes sur le sujet et dans De Pauw et Van Vaerenbergh (1981) et de la Noüe et De Pauw (1988) un examen critique des différentes dimensions de la question.

Dans le but d'évaluer l'efficacité d'un système de micro-algue (*Scenedesmus* sp) dans les conditions naturelles canadiennes, deux installations pilote ont été érigées dans deux villes différentes. Les expériences ont ainsi été menées simultanément à deux endroits de façon à pouvoir comparer les résultats du système opéré sous des conditions différentes (composition de l'effluent, opérateurs, climat, etc). Nous présentons ici les résultats relatifs aux taux d'épuration, à la quantité et à la qualité des biomasses produites.

Matériel et méthodes

Effluents

Les effluents secondaires utilisés pour les cultures de micro-algues sont ceux de la station d'épuration de la base militaire de Valcartier, située à 25 km au nord de la ville de Québec, et de la station d'épuration de Vaudreuil, sise à quelque 50 km à l'ouest de Montréal.

Dans les deux cas, les effluents reçus à la station subissent un traitement primaire par décantation, suivi d'un traitement secondaire par boues activées.

L'effluent de Valcartier est un effluent de nature surtout domestique, aucune contribution importante d'origine industrielle n'étant connue. L'effluent de Vaudreuil comporte surtout des eaux d'origine municipale, avec une contribution industrielle appréciable (entre autres la compagnie pharmaceutique Hoffman Laroche). La composition typique de ces effluents aparaît au Tableau 1.

Installations expérimentales

À chacun des sites (Valcartier et Vaudreuil) trois piscines circulaires hors terre de 5m X 1m, d'une capacité de.15 000 L , sont installées sur fond horizontal de sable. Le revêtement intérieur de ces piscines consiste en un plastique bleu, conventionnel pour ce type de piscine. Un système de bullage à l'air est monté afin de créer une turbulence suffisante et un apport continuel en CO_2 atmosphérique. À Valcartier, ce système est constitué de deux anneaux concentriques de tube perforé en plastique alors qu'à Vaudreuil trois anneaux ont été mis en place. L'air provient de suppresseurs; ceux-ci sont incorporés à la station d'épuration de Valcartier et fournissent de l'air à une température d'environ 40°C, tandis qu'à Vaudreuil ils sont indépendants et procurent de l'air à 25°C environ. Le volume horaire d'air utilisé correspond à environ 25 fois le volume d'effluent traité. Une alimentation en effluent secondaire, puisé à 0,5 m de la surface du décanteur secondaire, permet de remplir les piscines grâce à un système de pompage à grand débit.

Sur les lieux sont aussi présents des bassins de moindre volume (300 et 3000 L) servant de cyclostats pour la constitution des inocula, des systèmes de récolte des algues (centrifugeuse à bol à flux continu ou appareil à ultrafiltration) et, à chacun des sites, un laboratoire mobile pour les mesures et analyses immédiates.

Tableau 1. Caractéristiques des effluents et des installations à Valcartier et à Vaudreuil...

	Endroit	
	Vacartier*	Vaudreuil*
Solides totaux (mg/L)	186.0 - 331.0	150.0 - 546.0
Solides en suspension (mg/L)	8.0 - 3.0	1.2 - 11.0
NH_4^+ (mg N/L)	8.0 - 16.8	0.3 - 4.8
$NO_2^- + NO_3^-$ (mg N/L)	8.1 - 15.7	2.2 - 10.2
PO_4^{-3} (mg P/L)	2.3 - 3.4	0.1 - 2.2
Rapport N/P**	4.0 - 8.5	3.3 - 36.2
Moyenne	5.3	11.3
DBO_5 (mg/L)	8.0 - 13.0	1.7 - 26.0
pH	6.1 - 6.7	6.8 - 7.3
Turbidité (UNT)	1.0 - 3.7	--
Débit moyen (ML/d)	3.0	5.0
Population microbienne (10^3/mL)	1.5	2.3
Aération des cultures (vvm)***	0.35	0.50
Volume des bassins (L)	15 000	15 000
Volume des cultures (L)	10 000	10 000
Profondeur des cultures (m)	0.5	0.5

* Valeurs minimales et maximales courantes durant l'été.
** Base pondérale
*** Volume d'air équivalent à celui de la culture/minute

Modes de culture

Les cultures ont été opérées en vrac après inoculation à partir des cyclostats. Le compte cellulaire initial dans les piscines est de $1 \cdot 10^5$ cellules/mL environ.

Les cellules sont d'abord conditionnées en cyclostat (1 500 L; profondeur 20 cm) par inoculation de ceux-ci avec l'algue *Scenedesmus* sp. Les cyclostats sont opérés en semi-continu (taux de renouvellement de 0.50 d^{-1} avec de l'effluent secondaire décanté) jusqu'à l'atteinte, avant la dilution quotidienne, d'une population maximale et d'un épuisement total des nutriments. Les cellules sont alors jugées prêtes à servir d'inoculum pour les bassins de 10 000 L.

Mesures et analyses

Avant chaque utilisation de l'effluent dans les bassins, des aliquotes d'effluents sont prélevées pour la mesure des macronutriments (NH_4^+, NO_3^-, NO_2^-, PO_4^{-3}). De même, immédiatement après l'inoculation et des piscines et après mélange, des échantillons sont pris pour la mesure du compte cellulaire initial, de la densité optique, du pH et de la température. Des aliquotes d'effluent sont également prélevées pour la détermination des contaminants organochlorés et des métaux lourds.

Au cours des cultures, des échantillons sont systématiquement prélevés pour la mesure du pH, de la température, de la biomasse sèche et la vérification de la monospécificité des cultures. Lorsque le pic de biomasse et d'épuration d'au moins un des macronutriments est atteint, la culture est interrompue et les échantillons sont pris pour la mesure des divers paramètres précités. Les micro-algues sont alors récupérées, à l'aide d'une centrifugeuse à bol (Central Scientific, filtre 5 um) ou par ultrafiltration (Romicon, exclusion P M > 50 000) et les biomasses récoltées sont lyophilisées.

Les macronutriments (NH_4^+, NO_3^-, NO_2^-, PO_4^{-3}) sont dosées à l'aide d'un analyseur Technicon II (APHA *et al*, 1980).

La température est enregistrée sur un thermomètre à maximum et à minimun. Les données météorologiques de même que les données sur l'éclairement (ensoleillement relatif, rayonnement total quotidien et phase lumineuse) proviennent des aéroports de l'Ancienne-Lorette et de Dorval.

La biomasse est mesurée par pesée (balance Mettler H51AR), après séchage à 95°C pendant 24 heures de filtres Whatman GFC sur lesquels a été filtré un volume de 100 ml du milieu de culture.

Résultats et discussion

Aspects physico-chimiques

Les tableaux 2 et 3 montrent que le pH initial des cultures subit une variation non négligeable (6.6-8.2) et se situe en moyenne à 7.5 à Valcartier et 7.8 à Vaudreuil. Les pH ont tendance à être plus bas plus tard dans la saison à Valcartier, lorsque la température minimale est plus basse. On n'observe pas de comportement analogue à

Vaudreuil et les causes de cette différence, probablement à rechercher dans la composition des effluents, nous échappent. Plus significatif est le fait que le pH maximum enregistré est relativement constant d'une culture à l'autre et se situe à une valeur moyenne de 10.0 à Valcartier, ce qui indique une limitation certaine en CO_2. Il faut en conclure que l'aération utilisée était incapable de fournir le CO_2 atmosphérique à un taux suffisant à Valcartier. On a là une explication aux biomasses relativement faibles obtenues, les cultures ayant été limitées non pas en lumière, ce qui aurait été désirable, mais en CO_2. Il fait peu de doute qu'un enrichissement de l'air en CO_2 (5%) aurait mené à une productivité accrue (*cf* Tarifeno-Silva *et al*, 1982 par exemple). Un tel enrichissement est cependant coûteux et il convenait de mesurer la productivité dans les conditions les plus simples et économiques possibles.

Le problème semble moins aigu à Vaudreuil où le pH en fin de culture atteint 8.5 environ. On peut voir dans ces valeurs maximales différentes un effet du barbotage d'air plus vigoureux à Vaudreuil, ce qui résulte en un apport plus satisfaisant en CO_2. Le pH maximal de fin de culture atteint à Vaudreuil est cependant suffisamment élevé pour causer aussi une certaine précipitation du phosphate. Cette moindre biodisponibilité du phosphate à Vaudreuil explique en partie son caractère limitatif.

Tableau 2. Paramètres mesurés pour chacune des cultures menées au site de Valcartier

No de culture	Date de début	Durée totale	pH (moy)	T méd. (°C)	Ensoleillement rel.(%)	Rayonnement ($MJ \cdot m^{-2}$)	Phase lumineuse (%)
Q-1	22-juil.	139.0	8.3	22.9	34.6	21.0	62.1
Q-2	23-juil.	120.0	8.1	23.2	29.1	19.4	63.9
Q-3	24-juil.	147.0	8.2	23.5	23.5	17.7	62.9
Q-4	2-août	119.0	2.2	20.5	37.9	23.8	60.5
Q-5	4-août	288.0	8.2	20.2	26.2	19.3	59.7
Q-6	11-août	167.0	7.8	21.0	30.5	22.6	58.6
Q-7	13-août	148.0	8.1	21.0	30.6	21.6	59.2
Q-8	26-août	336.0	7.1	15.7	22.0	15.8	53.3
Q-9	30-août	262.0	7.8	15.5	19.4	14.4	54.2
Q-10	8-sept.	169.0	7.1	20.5	15.4	14.5	53.3
Q-11	27-sept.	384.0	6.7	11.9	18.7	11.7	47.8
Q-12	27-sept.	385.0	6.6	11.9	18.7	11.7	48.1
Q-13	27-sept.	386.0	6.6	12.3	18.7	11.7	48.1
moy. (c.v.,%)		234.5 (46.3)	7.5 (8.8)	18.5 (23.8)	25.0 (28.1)	17.4 (25.0)	56.3 (10.3)

Tableau 3. Paramètres mesurés pour chacune des cultures menées au site de Vaudreuil

No de culture	Date de début	Durée totale	pH (moy)	T méd. (°C)	Ensoleille-. ment rel.(%)	Rayonnement ($MJ \cdot m^{-2}$)	Phase lumineuse(%)
M-1	5-août	138.0	8.3	22.0	31.6	17.8	58.3
M-2	6-août	187.5	8.3	21.7	28.4	15.9	58.7
M-3	12-août	165.5	8.0	21.2	38.1	19.1	58.4
M-4	16-août	117.0	8.0	21.3	43.7	18.8	57.7
M-5	17-août	142.5	7.6	21.3	34.5	18.2	57.3
M-6	19-août	261.0	7.7	17.1	29.0	15.2	56.3
M-7	21-août	262.0	7.6	16.9	28.7	15.7	56.0
M-8	23-août	235.0	7.3	17.3	26.0	15.9	55.2
M-9	30-août	189.5	7.6	17.0	27.4	14.5	54.5
M-10	1-sept.	285.5	7.6	18.5	28.2	12.5	53.2
M-11	4-sept.	238.5	7.8	19.3	36.2	13.5	53.4
M-12	8-sept.	144.0	8.0	22.6	33.3	13.6	53.1
M-14	15-sept.	261.5	7.5	15.0	15.3	13.5	50.5
M-15	21-sept.	147.5	7.3	16.3	14.1	13.2	50.4
M-18	28-sept.	193.0	8.2	14.5	26.2	9.3	48.2
moy.		197.8	7.8	18.8	29.4	15.1	54.7
(c.v.,%)		(27.9)	(4.3)	(14.4)	(26.3)	(17.6)	(5.9)

Le rapport N/P mesuré pour les cultures effectuées à Valcartier se situe entre environ 3 et 7, ce qui explique la limitation observée en azote. En effet, il semble que *Scenedesmus* exige un rapport d'au moins 12 [de la Noüe *et al*, 1983]. Il convient de noter d'ailleurs que les rapports N/P mesurés durant les expériences ne semblent pas typiques de l'effluent de Valcartier où, au fil des ans, les rapports se maintiennent plutôt aux alentours de 15 (résultats non publiés).

Épuration

Le Tableau 4 présente l'élimination des nutriments obtenue pour chacune des cultures menés à Valcartier. On voit que l'enlèvement de l'azote total ne pose pas de problème, sauf dans les trois dernières cultures, qui ont, incidemment, été arrêtées prématurément. Bien que le NH_4^+ ait été complètement assimilé, il restait encore une bonne quantité de nitrates et nitrites dans ces trois cultures. On sait que les micro-algues utilisent préférentiellement la forme réduite de l'azote avant les formes oxydées (Syrret et Morris, 1963; Flores *et al*, 1980).

Tableau 4. Résultats relatifs à l'enlèvement de l'azote et du phosphore au site de Valcartier

No de la culture	Concentration initiale (mg/L)	Concentration finale (mg/L)	Épuration obtenue (%)	Temps** pour 95% (h)	Taux à 95 % *** (µg/L·h)
			N (NH_4^+, NO_2^-, NO_3^-)		
Q-1*	12.70	1.69	87		
Q-2*	13.12	0.74	94		
Q-3*	13.56	0.30	98		
Q-4	12.53	0.15	99	127	120.2
Q-5	13.22	0	100	268	46.9
Q-6	16.00	0.17	99	180	84.4
Q-7	19.61	1.14	94	149	125.1
Q-8	9.69	0	100	273	33.7
Q-9	12.29	0.28	98	217	53.8
Q-10	10.68	1.20	89	261	38.9
Q-11	9.04	2.51	72	470	18.3
Q-12	8.77	2.22	75	451	18.5
Q-13	8.26	2.10	75	458	17.1
			P (PO_4^{-3})		
Q-1*	3.01	2.45	19		
Q-2*	2.40	1.06	56		
Q-3*	2.86	0.79	72		
Q-4	2.50	0.61	76	152	15.6
Q-5	2.11	0.65	69	416	4.8
Q-6	2.06	0.71	66	256	7.6
Q-7	2.30	0.93	59	200	10.9
Q-8	2.19	0.74	66	621	3.4
Q-9	2.71	1.55	43	555	4.6
Q-10	2.69	1.33	51	394	6.5
Q-11	1.89	1.37	27	>1000	<1.3
Q-12	1.87	1.28	31	902	2.0
Q-13	1.91	0.96	50	655	2.8

* Manque de données pour établir la régression

** Durée requise pour une épuration à 95 %

*** Vitesse d'épuration moyenne jusqu'à 95 % d'épuration

Tableau 5. Résultats relatifs à l'enlèvement de l'azote et du phosphore au site de Vaudreuil

			N (NH_4^+, NO_2^-, NO_3^-)		
No de la culture	Concentration		Épuration obtenue (%)	Temps** pour 95% (h)	Taux à 95 % *** (µg/L·h)
	initiale (mg/L)	finale (mg/L)			
M-1*	2.17	0.41	81	157	13.1
M-2*	5.49	0.59	90	158	33.0
M-3*	4.79	0.24	95	148	30.7
M-4	8.30	1.14	86	124	63.6
M-5	4.13	0.28	93	150	26.2
M-6	6.91	0.81	88	250	26.3
M-7	4.40	0.16	96	229	18.2
M-8	3.35	0.11	97	202	15.8
M-9	8.96	0.23	97	148	57.2
M-10	7.45	0.16	98	227	31.2
M-11	3.56	0.55	84	204	16.6
M-12	7.24	3.00	59	230	29.9
M-14	7.28	1.66	77	274	25.2
M-15	10.39	2.53	76	189	52.2
M-18	3.29	0.35	89	237	13.2
			Phosphore (PO_4^{-3})		
M-1*	0.06		100	67	0.9
M-2*	0.48	0.015	97	104	4.4
M-3*	0.78		100	157	4.7
M-4	0.78		100	105	7.1
M-5	1.50		100	108	13.2
M-6	0.69	0.093	86	119	3.5
M-7	1.33		100	205	6.2
M-8	0.23	0.015	93	168	1.3
M-9	0.59		100	84	6.7
M-10	0.77		100	179	4.1
M-11	0.22		100	122	1.7
M-12	0.56	0.030	95	135	3.9
M-14	0.83		100	206	3.8
M-15	1.31	0.01	99	145	8.6
M-18	0.99		100	98	9.6

* Manque de données pour établir la régression ** Durée requise pour une épuration à 95 %

*** Vitesse d'épuration moyenne jusqu'à 95 % d'épuration

On doit cependant se rappeler que la durée des cultures triple presque, de la fin de juillet à la fin de septembre, Donc, même si l'élimination de l'azote total est toujours bonne, l'efficacité réelle du système varie beaucoup selon la période de l'année; le Tableau 4 illustre d'ailleurs cette augmentation de la durée requise pour obtenir un résultat donné d'épuration.

Lorsqu'on considère la durée requise pour un enlèvement de 95% du phosphore et de l'azote, l'élimination de l'azote se produit plusieurs heures auparavant; l'élimination du phosphore apparaît donc moins satisfaisante. Il semble qu'un rapport N/P un peu plus élevé - aux environs de 10 - aurait permis d'optimiser l'enlèvement pour le phosphore.

Le Tableau 5 présente les données d'épuration obtenues à Vaudreuil. En ce qui concerne l'azote total, la durée requise pour une épuration de 95% varie entre 124 et 174 heures. Compte tenu des faibles concentrations initiales, le taux d'épuration de l'azote est plus faible qu'à Valcartier, se situant en moyenne à 30,2 µg/L·h. Pour le phosphore, l'épuration est quasi-totale, probablement à cause des faibles concentrations initiales, ce qui entraîne des temps d'épuration plus rapides (entre 67 et 205 heures) qu'à Valcartier (> 150 heures). Le taux d'épuration du phosphore (5,4 µg/L·h) en moyenne) reste néanmoins comparable à celui de Valcartier (5,9 µg/L·h).

Lorsqu'on compare le temps requis pour l'épuration de l'azote et du phosphore (Tableau 4), l'élimination du phosphore précède celle de l'azote par un facteur de 1.5X, phénomène inverse par rapport aux résultats obtenus à Valcartier, mais qui soutient l'hypothèse qu'il ait été un des facteurs limitatifs dans la production de biomasses.

Production de biomasse

Les résultats relatifs à la productivité sont présentés au tableau 6.

La plus faible productivité de Valcartier peut être attribuée à plusieurs causes qui convergent toutes de façon cohérente pour expliquer cette différence marquée de productivité. La source d'azote étant principalement l'ammoniaque à Valcartier (tableau 1), et le pH des cultures étant plus élevé qu'à Vaudreuil (tableaux 2 et 3) , on admettra qu'une perte appréciable d'ammoniac (NH_3) ait pu se produire à Valcartier, réduisant par le fait même la biodisponibilité d'un nutriment, qui s'est d'ailleurs révélé limitatif. – Cette perte, bien que dépendante des conditions de culture, est évaluée à environ 30 % (Shelef *et al*, 1980; Pouliot *et al*, 1988) – En outre, la température a été plus basse, en moyenne, à Valcartier. Il est apparu aussi que la barbotage d'air a été plus intense à Vaudreuil qu'à Valcartier. Par ailleurs, une remise en suspension des cellules sédimentées a été régulièrement effectuée à

Tableau 6. Résultats relatifs à la productivité des cultures

a- Valcartier

No de la culture	Production algale		
	Concentration finale (mg/L)	Productivité (mg/L·h)	Biomasse totale (kg)
Q-1	62.6	0.451	0.626
Q-2	76.9	0.641	0.769
Q-3	82.9	0.564	0.829
Q-4	66.2	0.556	0.662
Q-5	141.4	0.491	1.414
Q-6	82.9	0.497	0.829
Q-7	87.9	0.593	0.878
Q-8	81.3	0.242	0.813
Q-9	68.0	0.260	0.680
Q-10	59.9	0.354	0.599
Q-11	59.0	0.154	0.590
Q-12	61.1	0.159	0.611
Q-13	62.7	0.162	0.627
Moyenne	76.4	0.394	9.927
(c.v.,%)	(28.8)	(45.6)	
b- Vaudreuil			
M-1	61.3	0.444	0.306
M-2	176.4	0.941	0.882
M-3	236.4	1.428	1.182
M-4	208.5	1.782	1.043
M-5	286.3	2.009	2.863
M-6	227.7	0.872	2.277
M-7	251.7	0.961	2.517
M-8	219.2	0.933	2.192
M-9	199.0	1.051	2.388
M-10	199.8	0.700	1.998
M-11	217.2	0.911	3.258
M-12	109.4	0.760	1.094
M-14	188.9	0.722	0.944
M-14	147.4	0.999	0.737
M-18	161.0	0.834	0.805
Moyenne	192.7	1.023	24.5
(c.v.,%)	(29.3)	(40.4)	

Vaudreuil, ce qui n'a pas été la cas à Valcartier. Enfin, les inocula ont été plus faibles à ce dernier endroit, condition qui se repercute directement sur la production, du moins pour *Oocystis* (Picard *et al*, 1980). On peut voir une confirmation de cet énoncé dans les résultats de la culture Q-5 qui, ensemencée avec un inoculum plus important, montre une productivité supérieure aux autres cultures de Valcartier. Outre l'importance de l'inoculum, la durée de l'incubation influence la production. Les résultats obtenus, à Valcartier et à Vaudreuil, sont assez comparables, tout compte fait, à ceux de Tarifeno-Silva *et al.* [1982] qui ont aussi cultivé *Scenedesmus* sur effluent secondaire avec barbotage d'air.

À Valcartier, la durée requise pour obtenir le pic de production reste à peu près constante de la fin de juillet à la fin d'août. À ce moment (culture no 8 et suivantes) la durée requise double, ce qui correspond à la chute enregistrée du rayonnement horaire moyen et de la température médiane. Ce résultat n'a rien de surprenant si l'on considère qu'à nos latitudes l'irradiance solaire maximum montre un pic accusé à la fin de juin et commence à décliner rapidement à partir du mois d'août (*cf.* Goldman, 1977).

La performance globale du système opéré, à Vaudreuil comme à Valcartier, n'approche pas la limite de 30 à 40 g (pds sec)/$m^2 \cdot d$ mentionnée par Goldman (1979b), résultat obtenu pour de courtes périodes dans des systèmes hautement contrôlés. Les résultats obtenus sont cependant du même ordre de grandeur, au pic de productivité (août), que ceux de 15-25g (pds sec)/$m^2 \cdot d$ obtenus par d'autres auteurs sur des laps de temps comparables aux nôtres dans des conditions de culture extérieure (Goldman, 1979a).

Il est intéressant de souligner que la productivité est malgré tout plus de trois fois supérieure à celle d'un champ de soja (bases de calcul: 50% de protéines dans la biomasse d'algue; soja: 600 kg prot./ha·an), résultat qui illustre le potentiel des cultures de micro-algues, même menées sous conditions non optimales (Berend *et al*, 1980; Pimentel *et al*, 1981).

Qualité des biomasses produites

La teneur en métaux lourds et en contaminants organo-chlorés dans les effluents et les biomasses a été mesuré. Les résultats, dont le détail a été présenté dans de la Noüe *et al*, 1986 et de la Noüe et Proulx, 1986, indiquent que:

- les concentrations des métaux lourds sont, par rapport aux normes préconisées, généralement acceptables dans les deux effluents (Cu et Zn sont toutefois en teneurs plus élevées que ces normes respectivement dans les effluents de Valcartier et de Vaudreuil);

- Il n'y a pas de bioaccumulation évidente des métaux lourds dans les cultures d'algues qui sont produites dans ces deux effluents durant 7 à 12 jours; bien que ce phénomène puisse survenir en laboratoire pour Pb, et, à plus faible échelle, pour Ni et Cu de l'effluent de Valcartier, il ne s'observe pas dans les cultures en vrac sur le site; ajoutons cependant que les teneurs en Mn et Zn des algues augmentent quelque peu (~20%) lorsque leur culture s'effectue dans l'effluent de Vaudreuil;
- les teneurs en contaminants organochlorés dans les deux effluents se sont révélées plus faibles que les seuils de détection des méthodes courantes qui permettent de les doser ou se sont situées généralement en dessous des normes recommandées par Environnement Canada (1980). Ils peuvent ainsi être considérés comme négligeables dans les deux effluents étudiés. De ce fait, les biomasses d'algues produites dans ces derniers ne sont pas contaminées par ces composés organochlorés.

Les données relatives à la composition biochimique des biomasses d'algues obtenues ne sont pas présentées ici, car elles ont été altérées par la présence insoupçonnée de sable dans les échantillons (apport héolien vraisemblablement). Cependant, ce type de données est bien connu pour *Scenedesmus* [Soeder et Binsack, 1978]. Bien que cette algue possède une teneur élevée en protéine (40-55 %), sa valeur alimentaire est cependant réduite par la présence d'une paroi cellulosique qui en diminue la digestibilité. Des régimes alimentaires à base de *Scenedesmus* ont déjà été avantageusement comparés à des régimes à base de soya, par de nombreuses études sur le Porc, le Poulet, le Rat et le Mouton par exemples [*cf* Becker, 1986]. Un traitement technologique approprié permet d'améliorer la digestibilité de biomasses d'algues telles que *Scenedesmus* (*cf* de la Noüe et De Pauw, 1988).

Soulignons en terminant, que l'algue *Scenedesmus* possède des propriétés thérapeutiques. Par exemple, il a été démontré que l'algue *Scenedesmus* procure un effet hypocholestérolémique marqué chez le Rat et qu'elle augmente remarquablement le taux de guérison de certaines maladies de la peau. Ces exemples et plusieurs autres sont décrit dans Becker 1986.

La récolte de micro-algues de la taille de *Scenedesmus* (5-15 µ) constitue encore de nos jours un obstacle majeur dans l'utilisation à grande échelle de ce système. Les technologies expérimentées depuis plusieurs années (centrifugation, filtration, floculation-flottation, etc) présentent des rendements insatisfaisants sur le plan de l'efficacité et des coûts d'opération (Benneman *et al*, 1980; de la Noüe et De Pauw, 1988). Cependant, le développement de nouveaux moyens, tel l'utilisation de floculants comestibles (Lavoie et de la Noüe, 1983; Mohn, 1988) ou l'intégration de systèmes aquicoles (de la Noüe *et al*, 1986) laissent présager la mise au point de solutions éventuelles.

Conclusion

Les résultats obtenus aux deux sites d'études montrent que les conditions locales d'opération, outre le contexte climatique, peuvent influencer considérablement le rendement du système. Il ressort également que ces micro-algues possèdent un pouvoir épurateur intéressant pour le traitement tertiaire des eaux usées et que la production de biomasse alimentaire est techniquement possible sur effluents secondaires d'origine domestique et urbaine, au moins durant les mois d'été La dépendance de la productivité à l'endroit des conditions climatiques et météorologiques indique cependant que la période favorable pour de telles cultures se situe à l'été, le rendement tombant assez rapidement avec une diminution de la température et des conditions météorologiques moins clémentes.

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INFLUENCE DES ORGANISMES FILAMENTEUX SUR LA TRAITABILITÉ BIOLOGIQUE DES EFFLUENTS

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INTRODUCTION

On associe souvent la teneur élevée de solides en suspension dans les effluents de stations d'épuration à la présence des bactéries filamenteuses. Smitt (1934) est probablement le premier à tenter de faire un rapprochement entre le "bulking", le type de bactéries filamenteuses et leur vitesse de croissance. Depuis, plusieurs auteurs ont tenté d'expliquer le phénomène du "bulking". Les travaux de McKinney et Gram (1956) ont montré la variété de la flore biologique et surtout la succession des espèces. Ceci a incité plusieurs à tenter d'identifier des organismes spécifiques associés au phénomène du "bulking": citons Curds et Cockburn (1970), Farquhar et Boyle (1971) ainsi que Strom et Jenkins (1984). Aucun n'est parvenu à identifier les correctifs nécessaires malgré une identification poussée des diverses espèces. D'autres ont examiné la question sur le plan de la configuration des bassins, (Pipes, 1969; Chudoba _et_ _al_., 1982; Lee _et_ _al_., 1982; Tomlinson et Chambers, 1984) ou sur le plan des conditions d'opération (Palm _et_ _al_. 1980; Wagner, 1984). Certaines recherches furent aussi faites pour tenter d'expliquer l'interaction entre la présence des bactéries filamenteuses et la séparation solide/liquide (Sezgin, 1980 et 1982). Ces recherches ont montré que la longueur des filaments était fortement corrélée aux caractéristiques de décantation des boues. N'ayant pas examiné cette question microbiologique sur un plan global, c'est-à-dire en tenant compte des facteurs opérationnels et de la qualité de l'eau à traiter, aucune relation de cause à effet ne pu être tirée.

Peu d'auteurs ont examiné la flore biologique en relation avec le rendement des procédés. Dias et Bhat (1964), Lighthart et Oglesby (1969) insistèrent sur l'importance d'examiner l'ensemble de la population biologique dans l'interprétation des résultats de traitement. Drakides (1980) signala que les analyses classiques étaient insuffisantes pour caractériser un régime transitoire. Dallas-Newton et Forster (1980) mentionnèrent que la présence de bactéries filamenteuses dans les boues activées était principalement le résultat des conditions auxquelles les systèmes étaient soumis et que celles-ci permettaient la croissance et la prolifération de ces microorganismes. D'autres ont vainement tenté de trouver un lien entre les membres de la communauté biologique et certains indices de rendement comme par exemple, les amibes et la transparence ou les flagellés et l'enlèvement de l'azote ammoniacal (Fujimoto et Arai, 1981). Cependant, Poole (1984) qui a étudié 3 types de stations d'épuration concluait que l'observation de l'ensemble de la communauté biologique permettrait la détection rapide de changements subtils de sa population tant en espèces qu'en nombre.

Ces diverses études nous laissent perplexes et nous amènent à poser la question suivante. Si les bactéries filamenteuses causent le "bulking", quels sont les facteurs provocateurs et quels sont les éléments de contrôle limitant l'effet perturbant que cause ce phénomène?

L'objectif du présent exposé est de poursuivre le débat soulevé par d'autres concernant l'influence des organismes filamenteux sur le rendement global d'un traitement par boues activées à la lueur de résultats d'opération obtenus en situation dynamique sur des effluents à haute charge et de composition variable (Zaloum, 1986).

MÉTHODOLOGIE EXPÉRIMENTALE

Le protocole expérimental de traitabilité et le mode d'opération des stations pilotes furent exposés ailleurs (Zaloum, 1987, Zaloum, 1988). Les eaux usées provenaient d'un abattoir situé à St-Louis de Gonzague dans la région de Montréal.

Pour obtenir des renseignements sur la population biologique, deux éléments d'information étaient requis:

(1) le nombre de microorganismes dans la biomasse;
(2) la diversité des groupes.

Ainsi, un examen microbiologique d'un nombre limité de conditions d'opération a été fait sur des échantillons de liqueur mixte prélevés des bassins d'aération. Les conditions choisies correspondaient à des temps de rétention hydraulique (TRH) et à des âges de boues (TRB) couvrant l'étendue des possibilités de traitabilité allant de combinaisons à faibles valeurs des deux paramètres à des combinaisons de valeurs élevées pour ceux-ci. Cet examen était fait sur des échantillons prélevés le dernier jour d'opération de la condition en question, ce qui donnait ainsi le maximum de temps disponible pour l'établissement d'une communauté biologique adaptée et représentative de cette condition. Il est à noter qu'une période de transition fut allouée entre chaque changement de condition d'opération afin d'assurer qu'un état d'équilibre soit atteint avant la prise de données. Les détails sont présentés ailleurs (Zaloum, 1988).

Dans la présente étude, une grille de comptage "Howard Mold" (1/10 mm de profondeur) a été utilisée. L'identification et le dénombrement ont été effectués avec un microscope à contraste de phases à Environnement Canada à Longueuil. D'après Stanier et al. (1970), cette méthode semble être la plus appropriée pour l'évaluation des populations biologiques. Elle permet un dénombrement des groupes de microorganismes alors que les autres méthodes sont restreintes à ce sujet. Elle détermine aussi un compte total qui inclut les fractions viable et non viable. Drakides (1980) a étudié le potentiel de cette méthode pour l'observation microfaunique des boues activées. Il concluait que la méthode était fiable et simple pourvu que le volume observé dépassait 10 mm^3.

Afin de surmonter la limitation temporelle inhérente à tout examen microbiologique, les échantillons étaient préservés au formol à 4%. Ceci rendait la méthode plus pratique car un dénombrement formel pouvait être reporté à une date ultérieure sans crainte de perdre l'information.

Avant d'aborder l'étude quantitative proprement dite, une étape préliminaire d'identification des microorganismes fut réalisée avant et après fixation au formol pour une estimation qualitative (présence - absence) et semi-quantitative (cote d'abondance).

Le dénombrement fut effectué en triplicata à 400x et ensuite à 100x.

À 400x, tous les organismes se trouvant dans le champ visuel sont dénombrés: 25 champs choisis au hazard sur la grille de comptage sont examinés.

À 100x, toute la surface de la grille est balayée pour recenser les gros organismes qui n'avaient pas été perçus à 400x.

Cette double observation permet le décompte des très petits organismes tels que Amoeba sp. à 400x et celui des plus grands et moins abondants tels que Podophyra sp. à 100x. Les résultats sont exprimés en nombre d'individus par ml.

En examinant les paramètres de l'eau brute et du surnageant simultanément dans des conditions d'équilibre, on a pu étudier l'interrelation entre la décantabilité des boues, l'indice du volume des boues et le dénombrement microfaunique ainsi que d'autres relations reliées à la traitabilité et au rendement globale du traitement.

Le logiciel SPSS a été employé pour le traitement statistique des données, lequel a été fait avec un ordinateur IBM-PC.

Le tableau 1 détaille la plage de conditions d'opération utilisées dans cette phase de l'étude en fonction du temps de rétention hydraulique (TRH) et de l'âge de boues (TRB).

Tableau 1: Grille expérimentale pour le dénombrement microfaunique.

TRH (J) TRB (J)	3	5	7	11	14	20
10			x			
15	x		x	x		
30						30
50	x	x			x	

RÉSULTATS ET DISCUSSION

Stabilité des populations

Pour obtenir une appréciation du changement qui pourrait se produire à l'intérieur d'une condition expérimentale, d'une semaine à l'autre, à cause d'une variabilité journalière de la charge organique, nous avons prélevé pour deux conditions différentes (TRH/TRB = 3/50 et 11/15) un échantillon par semaine. Le tableau 2 montre que, pour ces deux conditions étudiées, la distribution des classes ne semble pas avoir changé, sauf pour le développement d'amibes nues au deuxième prélèvement de la condition 11/15. Ces résultats nous permettent de croire, avec une certaine prudence, que les groupes microbiologiques sont relativement bien établis. Curds et Cockburn (1970) expliquent cette observation en indiquant que les protozoaires réagissent lentement aux variations hydrauliques et organiques de sorte que leur distribution en est peu affectée.

Tableau 2: Sommaire de l'identification microfaunique

Conditions expérimentales	Paramètres						
TRH/TRB	Charge organique appliquée	DBO_5 totale	DBO_5 filtrée	MVES effluent	Concentration de la biomasse	% du volume occupé par le lit de boues	Concentration des micro-organismes filamenteux
Jours	g/j	mg/l	mg/l	mg/l	mg/l	%	$\frac{\text{nombre}}{\text{mg biomasse}}$
3 / 15	186	14 (10)	-	40(38)	3 453	58	5,8 x 10^5
3 / 50	147	17 (17)	8,5(8,4)	112(240)	6 190	53	2,75 x 10^5
5 / 50	114	11 (9)	-	49(29)	3 655	27	1,23 x 10^5
7 / 15	64	5 (2)	3 (1)	20(8)	2 175	58	4,08 x 10^5
7 / 10	57,5	8 (3)	4 (1)	45(28)	980	19	5,9 x 10^5
11 / 15	30	-	-	31(14)	930	31	6,7 x 10^5
14 / 50	24,5	21 (6)	9 (2)	186(33)	1 440	19	2,7 x 10^5
20 / 30	20	-	6 (1)	157(30)	1 675	19	0,53 x 10^5

( ) = écart type

Effets des conditions d'opération sur la présence des organismes filamenteux

Ces bactéries étaient présentes dans toutes les conditions étudiées (tableau 2), indépendamment des rapports F/M dont les valeurs se situaient entre 0,06 et 0,43 g/g.j. Les concentrations d'oxygène dissous étaient maintenues généralement au dessus de 2 mg/l. L'examen des figures 1 et 2 montre que le nombre de bactéries filamenteuses était directement influencé par le TRH et par la DCO dans l'affluent. Contrairement aux résultats de la figure 1, on observait à chacun des deux niveaux étudiés de DCO brute une importante variation dans le nombre de microorganismes. En combinant le TRH et la DCO de l'affluent, on obtenait l'effet de la charge organique appliquée sur le nombre des bactéries filamenteuses (figure 3). Le résultat le plus surprenant fut l'absence d'effet évident de l'âge de boues sur le nombre de bactéries filamenteuses (figure 4). On constata ceci surtout pour deux ensembles de données aux âges de boues de 15 et 50 jours.

Relation entre la présence d'organismes filamenteux et le rendement du système de traitement

Zaloum (1986) développait une méthode rationnelle d'interprétation de l'évolution de la traitabilité par des surfaces de réponse et des régions d'isoréponse. Ces représentations réconcilient les écarts entre les prévisions théoriques et les constatations en systèmes réels.

Les figures 5 et 6 montrent les profils d'isoréponses pour la DCO filtrée et les solides en suspension dans l'effluent traité en fonction des paramètres d'opération, soit l'âge de boues et la charge organique appliquée.

En superposant les résultats des 8 conditions d'opération (excluant les répétitions) en fonction des surfaces de réponses (figures 5 et 6) nous ne pouvons faire aucun lien entre le nombre de bactéries filamenteuses, la DCO filtrée et les solides en suspension dans l'effluent. Également, en superposant les concentrations des organismes filamenteux au tableau 2, on constate qu'une diminution du nombre de bactéries filamenteuses coïncide sur la figure 6 avec un changement de région d'opération et avec un changement dans la qualité de l'effluent. Toutefois, à l'intérieur d'une même région d'isoréponse à la figure 5, par exemple aux âges

Figure 1: Influence du TRH sur le nombre d'organismes filamenteux présents dans la biomasse

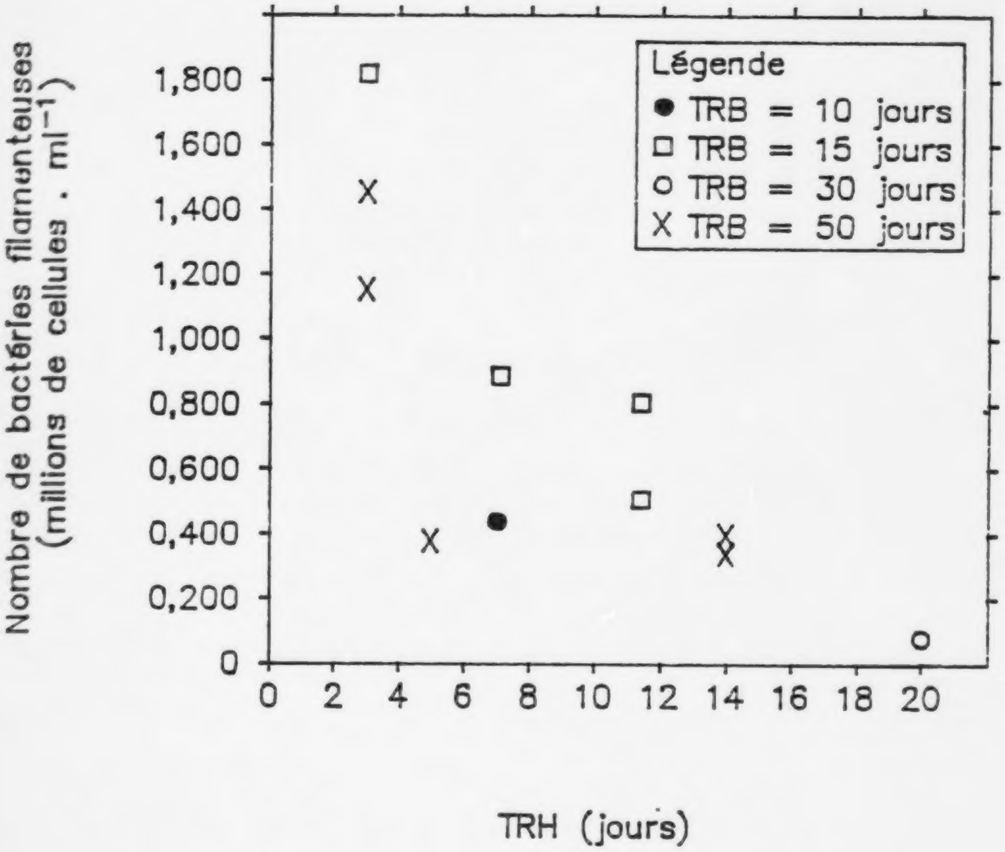


Figure 2: Influence de la DCO de l'eau brute sur le nombre d'organismes filamenteux présents dans la biomasse

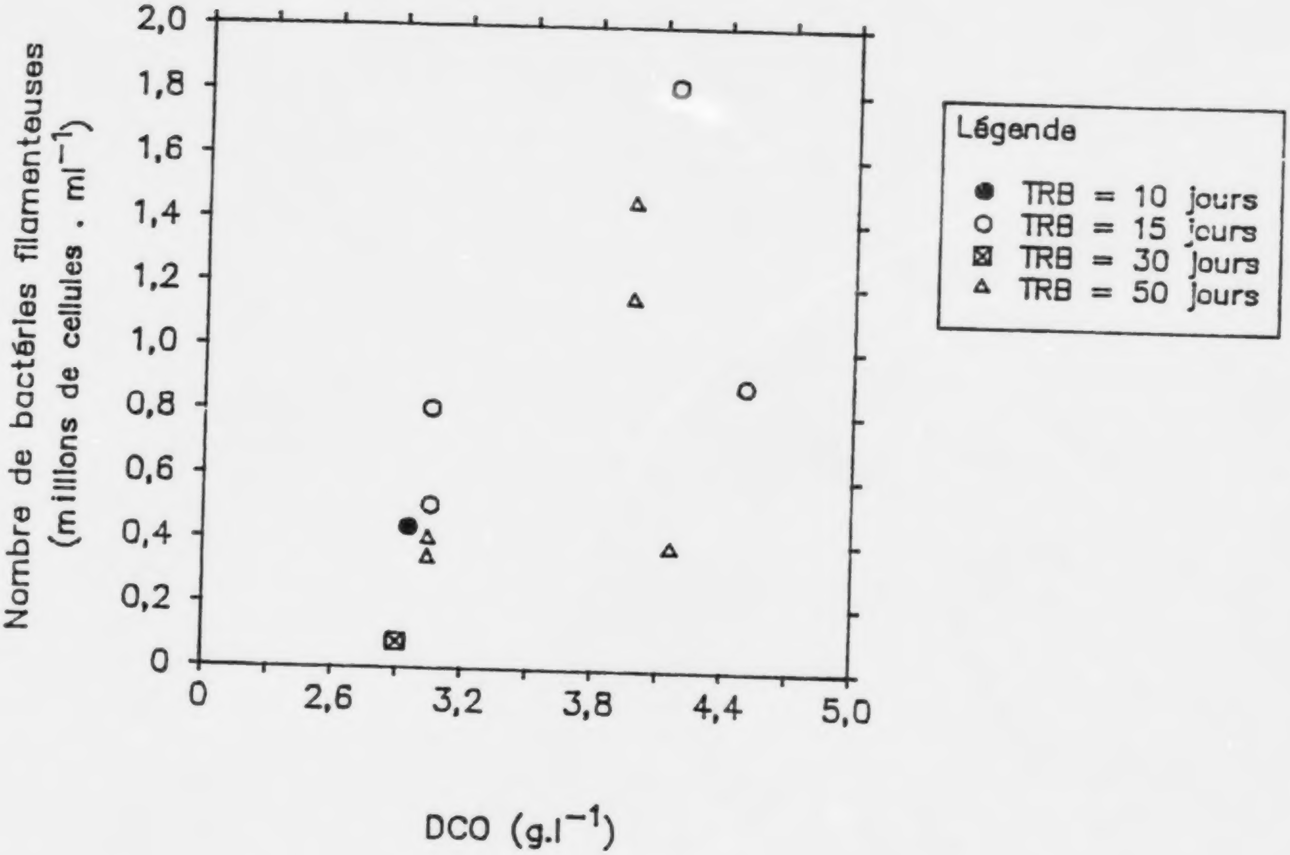


Figure 3: Influence de la charge organique appliquée sur le nombre d'organismes filamenteux présents dans la biomasse

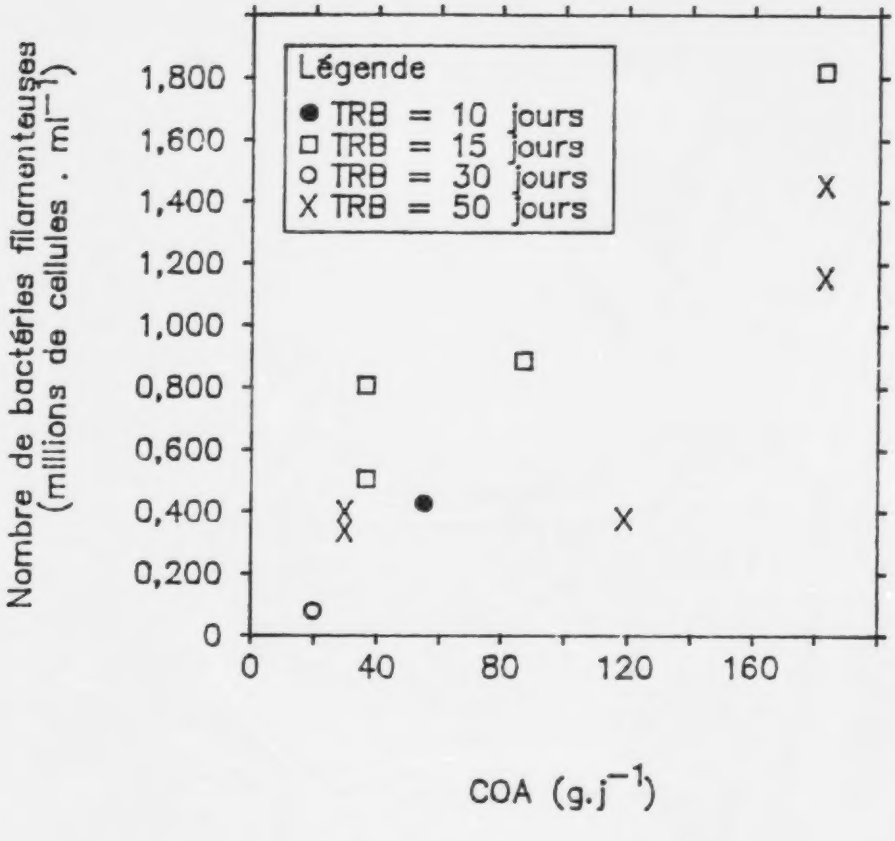


Figure 4: Influence de l'âge des boues sur le nombre d'organismes filamenteux présents dans la biomasse

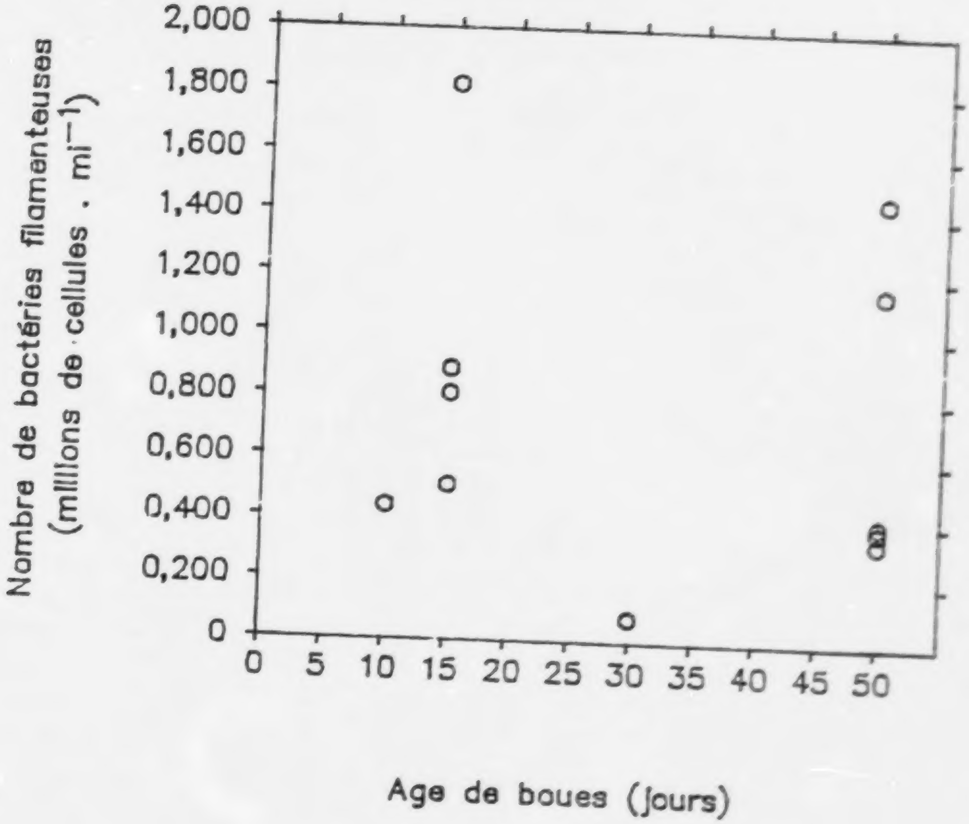


Figure 5: Surfaces de réponses de la DCO en fonction de la charge organique et de l'âge des boues

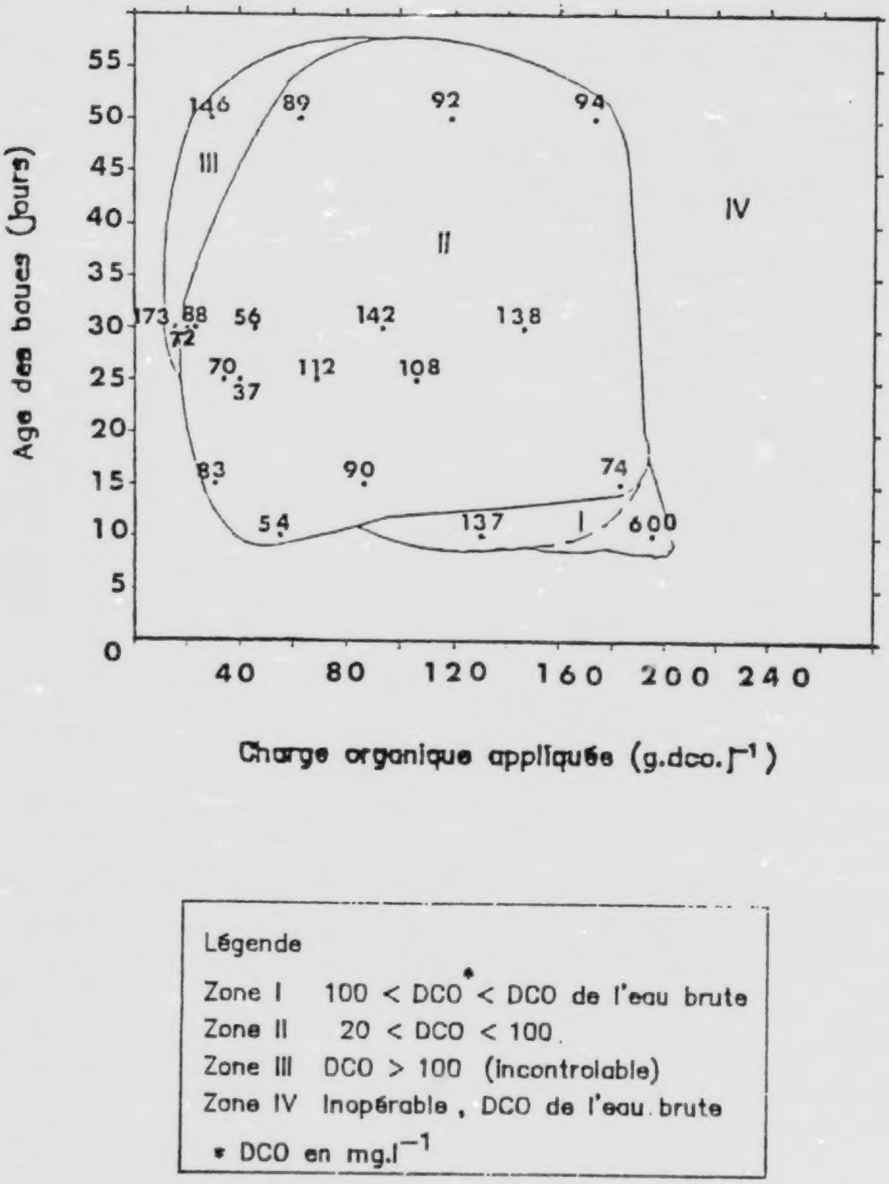
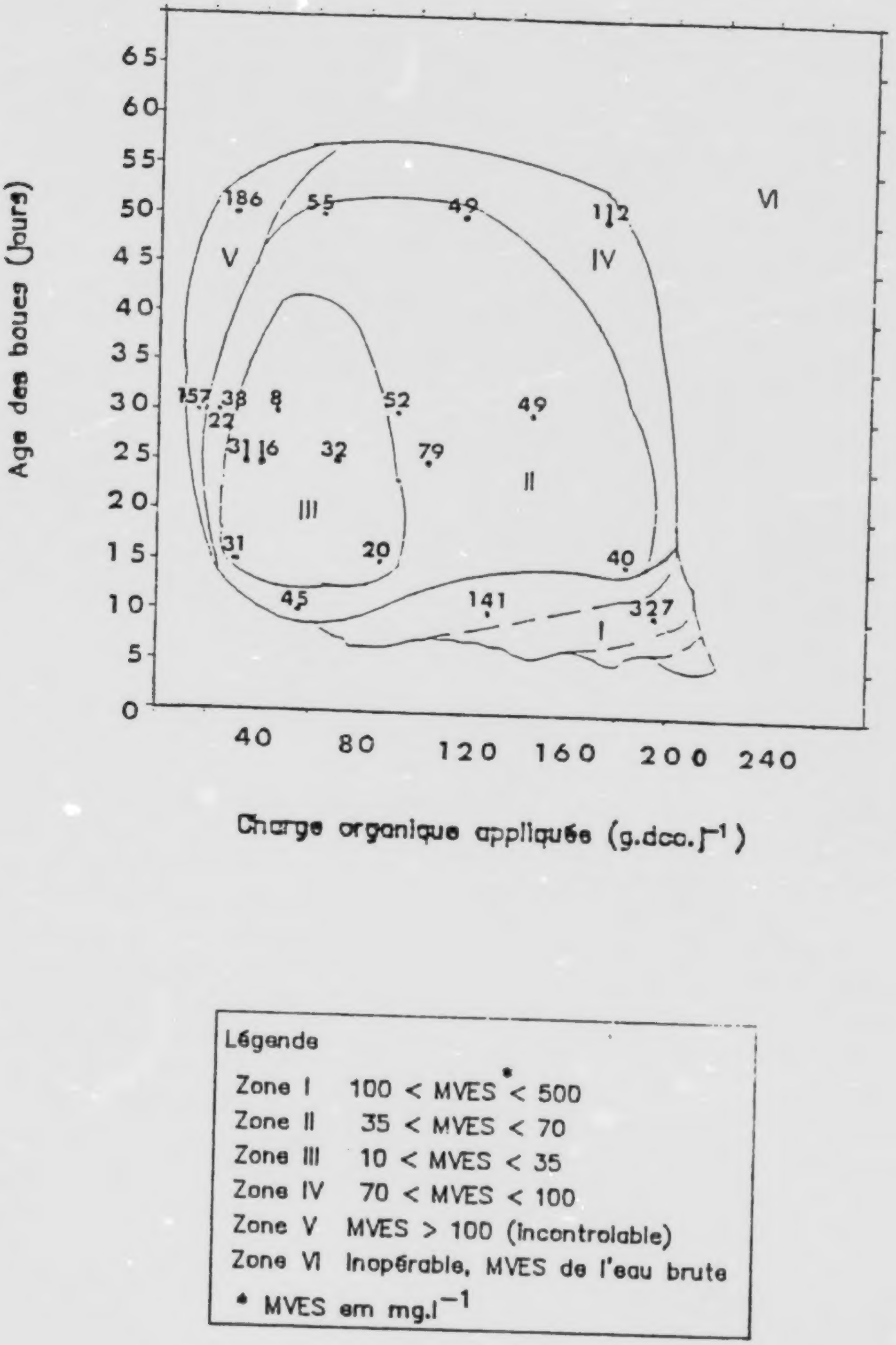


Figure 6: Surface de réponse des solides en suspension en fonction de la charge organique et de l'âge des boues



de boues de 15 et 10 jours dans la région II où la DCO filtrée de l'effluent demeure sensiblement la même, le nombre d'organismes filamenteux diminue appréciablement (conditions TRH et TRB 3/15 et 7/10). Il semblerait donc que le nombre de bactéries filamenteuses ne puisse pas être considéré comme un indicateur infaillible de performance. Leur présence doit cependant être prise en considération avec d'autres facteurs qui influencent la performance.

Le tableau 3 indique que le volume occuppé par la biomasse dans le réacteur augmente à un âge de boues donné avec la concentration de cette biomasse. Sur le plan opérationnel, plus le volume occupé par la biomasse augmente, plus la séparation solide/liquide devient difficile. Tel que montré au tableau 3, l'âge de boues et la charge organique enlevée influencent le volume occuppé par la biomasse durant le processus de décantation. La qualité de l'effluent commencerait à être affectée lorsque le lit de boues aura atteint un niveau de débordement ou d'entraînement de biomasse dans l'effluent. Ces constatations sont appuyées par Lee _et_ _al_. (1982). Ces auteurs ont observé que le nombre de microorganismes filamenteux était plus élevé quand, à débit constant, ils augmentaient la DCO de l'eau à traiter. Ils notaient que le gonflement des boues devenait évident quand la concentration de la biomase atteignait, dans leur cas, 3,5 g/l.

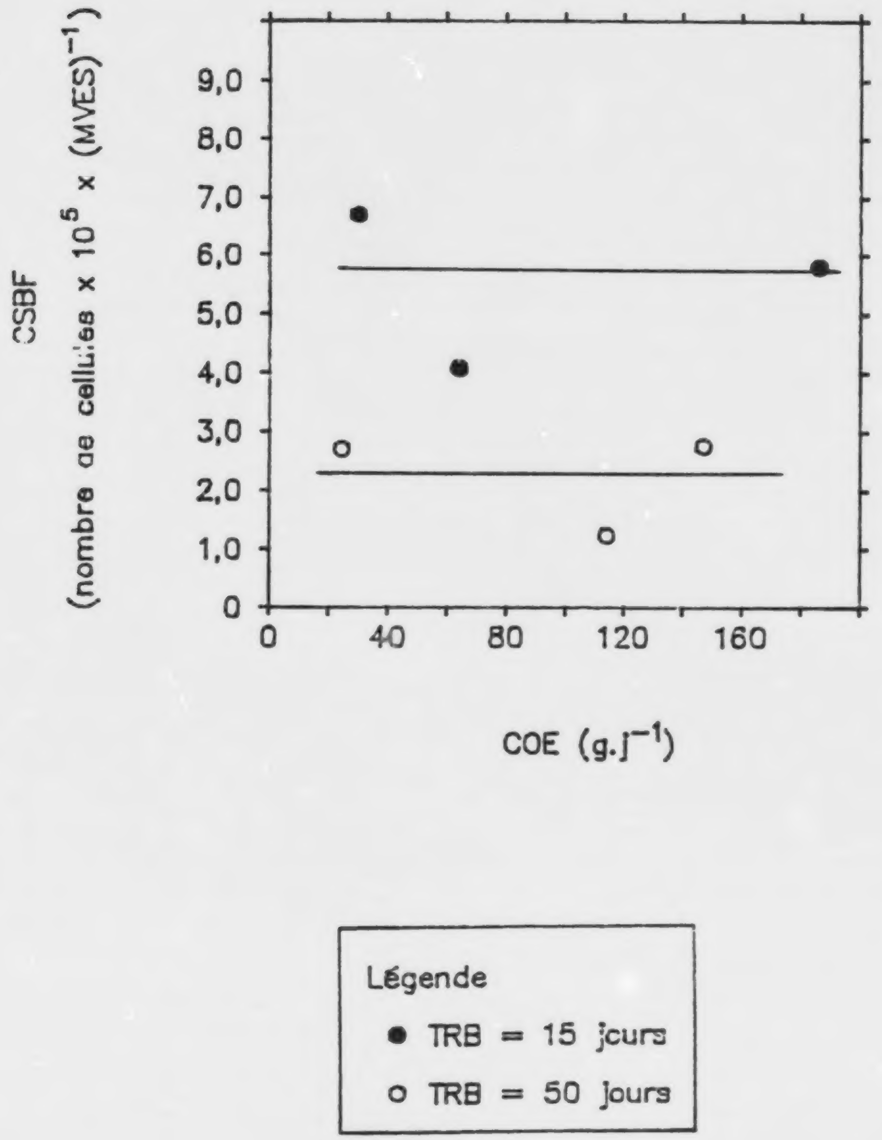
En exprimant le nombre de microorganismes filamenteux en fonction de la concentration de la biomasse, on observe à la figure 7 que le nombre de ces microorganismes par mg de biomasse dans le réacteur reste relativement constant en fonction de la charge organique enlevée à un âge de boues donné. Par contre, on remarque aussi que cette fraction diminue avec l'âge de boues. L'association du nombre de microorganismes filamenteux et de la biomasse par un rapport met en évidence l'effet de l'âge de boues sur l'efficacité d'épaississement et de séparation solide/liquide. Il faut noter, par ailleurs, que cette différence entre les deux niveaux d'âges de boues est plus faible qu'un ordre de grandeur. Cependant, nous croyons que cette différence reflète l'effet de ce paramètre d'opération. Ces résultats soutiennent que le nombre de bactéries filamenteuses est proportionnel à la concentration de la biomasse et qu'à un âge de boues donné, leur concentration par mg de biomasse reste constante. Ceci s'explique puisque le nombre de bactéries filamenteuses augmente avec la charge organique enlevée et que la concentration de la biomasse réagit exactement de la même manière envers ce paramètre.

Tableau 3: Résultats d'opération des conditions analysées pour leur contenu microbiologique

Organisme ind./ml.	Condition expérimentale et charge organique enlevée									
	(186)* 3/15**	(188) 3/50	(147) 3/50	(114) 5/50	(64) 7/15	(57.5) 7/10	(36) 11/15	(30) 11/15	(24.5) 14/50	(20) 20/30
Filamenteuse	$1,82 \times 10^{6}$	$1,2 \times 10^{6}$	$1,5 \times 10^{6}$	$3,8 \times 10^{5}$	$8,9 \times 10^{5}$	$4,4 \times 10^{5}$	$5,1 \times 10^{5}$	$1,82 \times 10^{5}$	$3,4 \times 10^{5}$	$7,9 \times 10^{4}$
Flagellés	$2,1 \times 10^{4}$	$5,2 \times 10^{5}$	$1,4 \times 10^{5}$	$0,83 \times 10^{4}$	$1,8 \times 10^{5}$	$2,4 \times 10^{5}$	$3,7 \times 10^{4}$	$2,1 \times 10^{4}$	$7,5 \times 10^{4}$	$1,4 \times 10^{3}$
Amibes nues	$1,9 \times 10^{4}$	$3,3 \times 10^{4}$	$9,3 \times 10^{4}$	$1,25 \times 10^{4}$	0	$3,7 \times 10^{4}$	0	$1,9 \times 10^{4}$	$4,4 \times 10^{4}$	$5,7 \times 10^{4}$
Amibes Testacées	33	5×10^{2}	$6,4 \times 10^{2}$	$1,1 \times 10^{4}$	$6,9 \times 10^{3}$	11	0	0	$5,2 \times 10^{2}$	$5,6 \times 10^{3}$
Ciliés fixés	$2,5 \times 6^{3}$	$1,8 \times 10^{3}$	$1,3 \times 10^{3}$	$5,5 \times 10^{3}$	$1,7 \times 10^{3}$	$1,6 \times 10^{2}$	0	0	$1,7 \times 10^{3}$	$6,2 \times 10^{2}$
Ciliés libres	44	0	300	56	111	$1,3 \times 10^{4}$	0	778	$1,7 \times 10^{4}$	$6,9 \times 10^{3}$
Rotifères	$2,3 \times 10^{2}$	$2,5 \times 10^{4}$	$4,3 \times 10^{3}$	$2,4 \times 10^{4}$	$9,7 \times 10^{3}$	$5,9 \times 10^{2}$	0	89	$5,8 \times 10^{4}$	$4,99 \times 10^{4}$
Nématodes	89	67	22	0	33	11	0	0	0	0
Algues	$4,9 \times 10^{4}$	$2,5 \times 10^{4}$	$2,8 \times 10^{4}$	$3,7 \times 10^{4}$	$1,9 \times 10^{4}$	$5,5 \times 10^{4}$	$4,6 \times 10^{4}$	$8,9 \times 10^{4}$	2×10^{5}	$2,3 \times 10^{4}$
Hydracariens	0	0	22	0	78	0	0	11	0	0

* charge organique enlevée, g/j, pour la journée de l'identification   ** TRH/TRB, jours.

Figure 7: Relation entre la concentration spécifique des organismes filamenteux (CSBF) et la charge organique enlevée (COE)

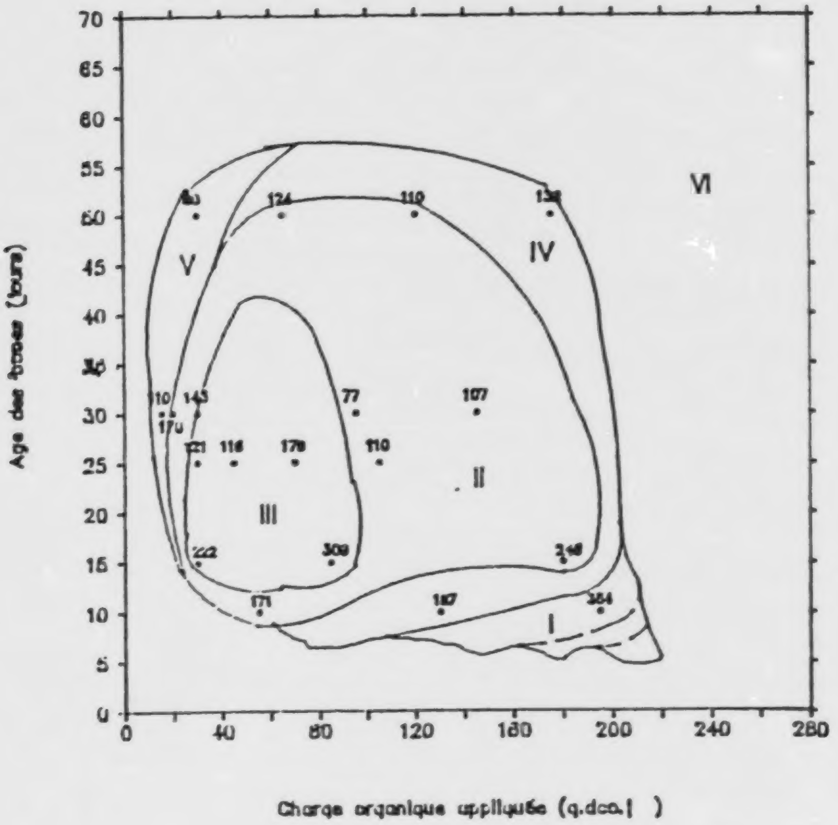


On montre à la figure 8 les valeurs de l'indice du volume des boues (IVB) superposées sur la figure 6 où sont définies les régions d'isoréponse pour la teneur en matières volatiles en suspension de l'effluent. En comparant les valeurs respectives des deux figures dans les régions d'isoréponse II et III et en examinant les valeurs d'IVB dans chaque région d'isoréponse, on ne décèle pas de relation entre l'IVB et la teneur MVES de l'effluent. Le fait que l'indice du volume des boues puisse varier sur une grande plage à l'intérieur d'une même région montre bien qu'il ne reflète pas fidèlement les effets des paramètres d'opération (TRB et charge organique enlevée) sur la biomasse. Il devient donc impossible en se basant sur cet indice de modifier l'opération dans le but de maintenir une concentration désirée de solides dans l'effluent. Il est nécessaire d'indiquer avec la mesure de cet indice les conditions d'opération et la hauteur du lit de boues durant la phase de séparation afin d'établir la probabilité d'entraînement de biomasse dans l'effluent. Il est à noter également qu'une boue défloculée (région V) peut avoir un IVB comparable à celui d'une boue sur le point de déborder du réacteur (région IV) ou à celui d'une boue d'une opération dans la région II et III. De même, sans points de repère tels que l'âge de boues et la charge organique, une biomasse dans la région I aurait un IVB comparable à celui d'une biomasse provenant d'une opération dans la région II ou III. De façon générale, l'IVB peut donner une indication du degré de compaction de la biomasse et servir de guide de comparaison pour une biomasse donnée.

L'examen des résultats de traitabilité montre que l'IVB était plus élevé aux TRB de 10 et 15 jours qu'aux TRB de 30 et 50 jours. Ces valeurs d'IVB corespondent aux concentrations spécifiques des bactéries filamenteuses, lesquelles étaient également plus élevées à l'âge de boues de 15 jours qu'à celui de 50 jours (voir figure 8). Cet effet de compaction, relié à l'opération à un âge de boues élevé par rapport à un âge de boues faible, montre que la boue biologique se concentre mieux lorsque le nombre de microorganismes filamenteux par unité de biomasse diminue. Une telle relation n'a pas été trouvée quant à la qualité de l'effluent. En pratique, on observe que l'exploitation des stations d'épuration à un âge de boues élevé minimise la production de boues et améliore la compaction et le degré de siccité des boues éliminées.

Clairement, la mise en relation par le concept de surfaces de réponse et d'isoréponse semble expliquer certaines observations et pratiques courantes dans l'exploitation des systèmes d'épuration.

Figure 8: Relation entre l'indice du volume des boues (IVB) et la teneur en solides volatils en suspension dans l'effluent



Légende

Zone I 100 < MVES* < 500
Zone II 35 < MVES < 70
Zone III 10 < MVES < 35
Zone IV 70 < MVES < 100
Zone V MVES > 100 (Incontrolable)
Zone VI Inopérable, MVES de l'eau brute

* MVES em $mg.l^{-1}$

À ce propos, Knocke (1986) présente des résultats d'essais confirmant l'influence de l'âge de boues sur l'épaississement de la biomasse.

L'examen des groupes de protozoaires (flagellés, amibes nues et testacées, ciliés libres et fixés) a révélé que l'association de ces organismes à une haute qualité de traitement par rapport à la matière organique soluble n'assurait pas une teneur faible de solides en suspension dans l'effluent (voir tableaux 2 et 3 et figures 5 et 6). Leur manifestation répond aux conditions imposées, leur nombre n'étant pas influencé au delà d'un âge de boues de 15 jours; il s'en suit que leur présence ne peut être identifiée qu'avec des conditions favorables à une bonne traitabilité. L'optimisation du rendement dépendra uniquement de la manipulation des conditions d'opération. Les surfaces de réponse montrent la direction du changement dans l'évolution de la traitabilité suite à une manipulation donnée.

On a également observé qu'aux âges de boues plus faibles que 10 jours, les rotifères étaient absents. Cette absence coïncidait avec une traitabilité réduite et une opération dans la région I des figures 5 et 6. Ainsi, leur présence est plus un indice que les conditions écologiques sont suffisamment favorables pour assurer leur manifestation qu'une garantie de bon rendement. De plus, leur présence n'est pas identifiable avec un mode d'opération optimal, leur évolution étant une réponse aux conditions d'opération en vigueur. De fait, nos résultats indiquent que les rotifères et les nématodes sont présents à haute et à faible charges et que leur manifestation dépend plus de l'âge de boues que d'autres facteurs.

CONCLUSION

Le dénombrement microbiologique réalisé dans cette étude n'a pas permis d'établir des relations de cause à effet pour le rendement. La concentration des organismes évoluait en fonction des conditions d'opération.

L'ensemble des données précédentes nous amène à conclure que la présence des organismes filamenteux ainsi que l'évolution des autres groupes microfauniques sont en fait des réponses aux conditions d'opération tels que l'âge de boues et la charge organique enlevée. Ces microorganismes font partie d'un écosystème contrôlé par ces paramètres d'opération. L'absence de relation entre l'enlèvement de la matière organique soluble et l'évolution des groupes microfauniques peut découler du fait que cette matière soluble est principalement éliminée par les bactéries hétérotrophes. Les mécanismes qui font qu'une biomasse ait plus ou moins de bonnes propriétés biofloculantes résultent de l'équilibre qui s'établit entre les différents groupes de la communauté microbiologique.

L'analyse du rôle des microorganismes filamenteux à l'aide de surfaces de réponse révèle que la présence ou le nombre de ces microorganismes n'est pas en soi un indicateur de performance. Nous avons trouvé que la concentration de microorganismes filamenteux par mg de biomasse demeurait relativement constante à un âge de boues donné lorsque la charge organique variait. La compaction de la biomasse semblait directement reliée à cette concentration spécifique. Cependant, la qualité de l'effluent en matières en suspension n'y était pas reliée.

L'indice de volume des boues, qui peut donner une mesure de compaction des boues, était quasi-indépendant, dans la majorité des cas, de la teneur en MVES de l'effluent.

Il n'est toutefois pas exclu que des conditions particulières d'opération ou de déséquilibre dans l'alimentation puissent favoriser une croissance accélérée de certains organismes filamenteux détériorant ainsi la qualité du rendement. L'identification de tels organismes devrait être faite en tandem avec une analyse structurée de l'effet des conditions d'opération et de la qualité de l'affluent afin de pouvoir appliquer les correctifs appropriés et rétablir le bon rendement de la station d'épuration.

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POTENTIAL HEALTH RISKS AMONG SEWAGE WORKERS DUE TO AIRBORNE MICROORGANISMS, PARASITES AND ENDOTOXINS

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ABSTRACT

A study was conducted at a sewage treatment plant in Toronto, Ontario to assess the workers' exposure to airborne bacteria, endotoxins and parasites. The surveys were carried out in the dewatering section of the plant when: a) no machines were in operation; b) only one belt press was operating; and c) the building was functioning at maximum capacity with two vacuum filters and one belt press in operation. Air samples, as well as samples of wet and dried sludge and plant water, were analyzed for total heterotrophic bacteria, fecal coliforms, *Escherichia coli*, *Pseudomonas aeruginosa* and endotoxin levels. The same samples were also examined for parasites such as Acanthamoeba, Naegleria, Giardia, and Cryptosporidium.

The bacteriological results showed that the levels of total heterotrophic bacteria, fecal coliforms, *E. coli* and *P. aeruginosa* in the wet and dried sludge and in the plant water differed very slightly over the three sampling days. Consistently high levels of endotoxin were found in the wet and dried sludge, whereas lower concentrations of endotoxin were present in the water during the same period.

On the sampling day when the entire dewatering plant was shut down, no fecal coliforms, *E. coli*, or *P. aeruginosa* could be isolated from the air. Furthermore, it was not possible to isolate endotoxins from the air in the building within the 30 minute sampling time.

With one belt press in operation, three of the 14 air samples yielded fecal coliforms and *E. coli*. *P. aeruginosa* was not found in any of the air samples. Two of the seven sites tested yielded endotoxin from the air.

Finally, when the two vacuum filters and one belt press were operating, one air sample produced fecal coliform and *E. coli* counts and two air samples were positive for *P. aeruginosa*. The increased activity in the plant could be noted by the fact that four of the eight samples were positive for endotoxin.

Free living amoebae were isolated only from sludge samples. None of the air or water samples yielded parasitic organisms. Four of the 24 amoebic isolates were identified as *Naegleria* spp. while the remaining were identified as members of the *Acanthamoeba* group. Intestinal helminths and protozoa of medical importance were not detected.

INTRODUCTION

The increased number of municipal sewage treatment products, such as compost, and the possible contamination of lake water with such products have been the concern of many urban areas, especially those located near lakes. The sewage treatment process eliminates or destroys some pathogens but there are others that are resistant to such treatment.

When sewage sludge is being processed, microorganisms can be released into the air. If the number of microbes per unit volume of air is high, inhalation of such aerosols may present a potential health threat. Sewage worker's syndrome has been described among sewage plant workers handling sewage sludge which was dried and became aerosolized during transportation. Influenza-like symptoms were present among 30 - 50% of the workers (Mattsby and Rylander, 1978). These symptoms are typical of those expected after inhalation of airborne endotoxin (Rylander and Snella, 1983). Other studies have shown that gastrointestinal disturbances among sewage workers may be related to exposure to enterotoxin secreted by Gram-negative bacteria (Lundholm and Rylander, 1983).

Parasites have also been shown to survive sewage processing. Cryptosporidium, a newly recognized intestinal protozoan, has been recognized as an important cause of diarrhea in man. This organism has been recovered from lake water and sewage,contaminated river water (Sterling et al, 1987) and from drinking and waste waters (Sterling et al, 1987). In addition, Naegleria and Acanthamoeba have been commonly found in soil, fresh water and sewage sludge.

The purpose of this study was to examine the air and water, as well as the wet and dried sludge in the dewatering building of a sewage treatment plant in Toronto to determine bacterial, endotoxin, mold and parasitic levels. Samples were collected from various locations in the building on days when the plant was fully operating, partially operating, or not operating.

MATERIALS AND METHODS

Several different sampling sites were chosen in the dewatering building of the sewage treatment plant located in Toronto, Ontario. Air, processed wet sludge, dewatered sludge (dry sludge) and plant water samples were taken from different sites in the building as shown in Figure 1. Samples were collected from the various locations in the building when the building was fully operating, partially operating and not operating. As controls, air samples were obtained from an office at the University of Toronto, and City of Toronto tap water was was collected.

Airborne bacteria were collected with a 2 stage Anderson Sampler equipped with petri dishes containing mPA for isolation of Pseudomonas aeruginosa (APHA, 1985), mTEC for enumeration of fecal coliforms (Dufour and Cabelli, 1975) and R2A agar for quantitation of total heterotrophs (APHA, 1985). Fecal coliform isolates were confirmed as Eshcerichia coli using standard biochemical reactions. Results are reported as concentration of bacteria per cubic meter of air.

Midget impingers were used to assess the level of endotoxin in the air. Material used in this analysis was baked at 180°C for 4 hours to render it pyrogen-free. Ten mL of pyrogen-free water was added to impinger vials prior to use. The impinger was run for 30 min at a flow rate of 1 L per min. The resulting air-bubbled samples were serially diluted in pyrogen-free water and endotoxin levels determined using the Limulus amebocyte lysate (LAL) assay (Assoc. Cape Cod). A 0.1 mL sample of LAL was added to 0.1 mL of sample or diluted sample in 10 x 75 mm test tubes. Tubes were incubated in a 37°C water bath for exactly 1 hour. The tubes were then inverted and the presence of a stable gel clot was regarded as a positive reaction. The test was run with a sensitivity of 0.125 EU/mL. Control

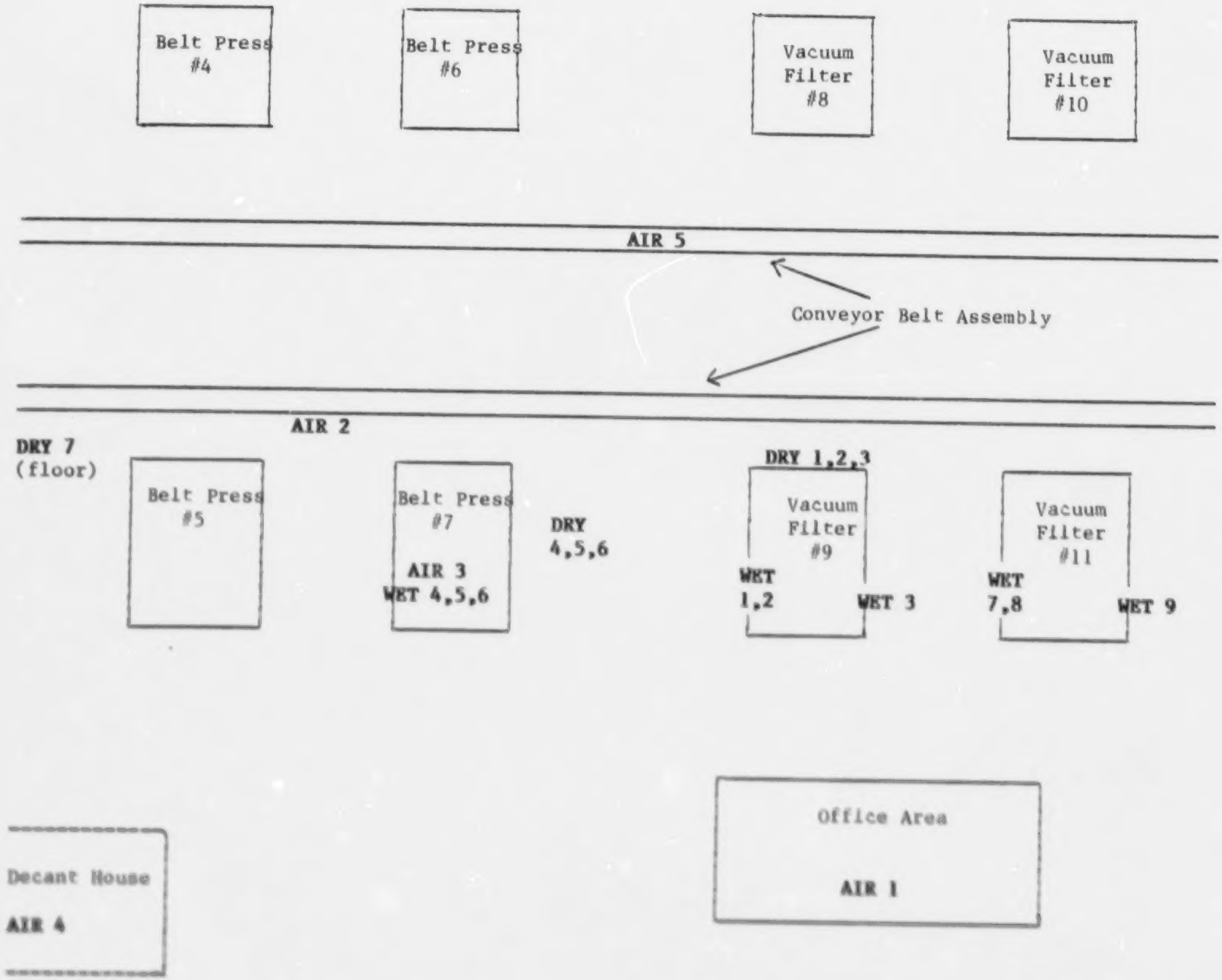
Belt Press #4
Belt Press #6
Vacuum Filter #8
Vacuum Filter #10
AIR 5
Conveyor Belt Assembly
AIR 2
DRY 7 (floor)
Belt Press #5
Belt Press #7
AIR 3
WET 4,5,6
DRY 4,5,6
DRY 1,2,3
Vacuum Filter #9
WET 1,2
WET 3
Vacuum Filter #11
WET 7,8
WET 9
Decant House
AIR 4
Office Area
AIR 1

Figure 1: Schematic diagram of sampling sites in the dewatering building.

tests were run to ensure the accuracy and sensitivity of the tests. Results are reported as ng endotoxin per m^3 air.

Plant water was examined for the presence of P. aeruginosa, total heterotrophs and fecal coliforms using standard membrane filter techniques (APHA, 1985) using mPA, R2A and mTEC media respectively. Filters obtained from the fecal coliform procedure were subjected to the urease test, as described by Dufour and Cabelli (1975), to determine E. coli levels. Endotoxin levels were assessed using the LAL assay.

Wet and dry samples were collected in sterile pyrogen-free bags. One gram of the sludge sample was diluted in phosphate-buffered water (APHA, 1985) and mixed thoroughly. Serial dilutions of the stock suspension were made and the appropriate dilutions membrane filtered. Bacterial isolation media were mPA, mTEC, and R2A agar. Filters from the mTEC procedure were subjected to the urease test to quantitate E. coli levels.

Endotoxin levels were determined in the following manner. A 0.1 g amount of the sludge material was added to pyrogen-free test tubes containing 9.9 mLs of pyrogen-free water. The sample was vortexed periodically for 30 min followed by centrifuging at 1000 x g for 10 min. One mL of the supernatant fluid was removed and serially diluted in pyrogen-free water. Appropriate dilutions were tested for the presence of endotoxin using the LAL lysate assay. Results are reported as µg of endotoxin per gram of sludge.

Approximately 20 g of each wet sludge sample was mixed with 20 mL of a saline-triton solution and the suspension was filtered through two layers of gauze. A portion of the filtrate was examined by zinc sulfate flotation technique for helminth's eggs, Giardia and Cryptosporidium species (Melvin and Brooke, 1985). The remaining filtrate was used to isolate Acanthamoeba and Naegleria in Amebic agar plates precoated with E. coli (Phillips and Nash, 1985).

For dry sludge samples, 10 g of the sample was first soaked in 20 mL of a saline-triton solution for 24 to 48 hours at room temperature. The soaked sludge was then processed in the same manner as the wet sludge.

Water samples were processed by filtering approximately 50 mL of each water sample through a 5 um pore sized filter. The material on the filter was then collected by washing it in 10 mL Page's amebic solution in a test tube. After centrifugation for 10 min, a portion of the sediment was used to culture free-living amoeba in agar plates and the remaining sediment for preparation of smears on the microscopic slides. The smears were stained with a modified Ziehl-Neelsen stain for Cryptosporidium spp. and with hematoxylin for Giardia spp.

Air samples in distilled pyrogen-free water were examined for free living amoebae only. The growth of free-living amoebae was examined microscopically on the surface of the culture plates. If the cultures were positive, transfers of the amoebae to fresh plates were performed every 2 to 3 days until the cultures were free of bacterial contamination.

The identification of the isolated amoebae was based on an exflagellation test in the distilled water and also on their ability to grow at 45°C. The pathogenicity of the isolated amoebae was tested on white mice about 2 - 3 weeks old, weighing 5 - 10 g. The mice were infected intranasally under ether anaesthesia. The volume of the inocula was 0.02 - 0.03 mL and the count of amoebae was about 20,000 to 30,000. Those mice which died three days to several weeks after inoculation were autopsied and the brains and lungs examined for the presence of amoebae by staining the tissue smears with hematoxylin.

RESULTS

The level of airborne bacteria present in the working environment was observed to vary with each sampling event (Table 1). When the building was not in operation, the concentration of bacteria ranged from 281 colony forming units (cfu) per cubic meter of air to 578 cfu/m^3. Upon further analysis of the colonies on each plate, approximately 62 -67% of the total colony forming units were identified as being Gram-negative bacteria. For this analysis, a value of 67% will be used as the proportion of Gram-negative bacteria on each plate. Only one respirable air sample, Belt Press 3RS, had a concentration of Gram-negative bacteria greater than 1000 cfu/m^3. The level was calculated to be 1830 cfu/m^3. In the control air sample, the density of Gram-negative bacteria was 21 cfu/m^3.

Higher concentrations of airborne bacteria were encountered when the building was in full operation as compared to the other two sampling events. Greater concentrations of bacteria were found where aerosol mists were generated, namely sites Air 2, Air 3, Air 6 and Air 8.

The concentration of mold, cfu/m^3, ranged from 0 (staff room air sample) to a maximum of 278 detected from the sample taken in the belt press (Air 3). The highest levels of mold, 278 and 208 cfu/m^3 were encountered during the period when the building was not in operation. In comparison, the highest level of mold detected during full operation was 108/m^3 air (Table 1).

The concentration of heterotrophic bacteria in the sludge samples ranged from 10^5 to 10^8 cfu per gram of sludge (Table 2). The levels did not vary considerably between wet and dry samples taken from the same site. A dry sludge (Dry 7) sample of indeterminate age, present on the floor, had an average heterotrophic bacterial concentration of 1.3 x 10^8 cfu/g over the three sampling periods.

The level of heterotrophic bacteria detected in the effluent water samples did not follow any specific pattern. Water sample 2 had approximately 10^4 cfu/100 mL over the three sampling events while the other samples fluctuated in bacterial levels by as much as 10^5 cfu/100 mL (Table 2). In comparison, city tap water had less than 1 cfu per 100 mL sample.

Pseudomonas aeruginosa was detected in only one air sample taken when the system was in full operation (Table 3). Fifty percent of the wet sludge samples were found to contain *P. aeruginosa* at levels ranging from 1.0 x 10^2 to 7.0 x 10^2 organisms per gram of sludge. The concentration of this microorganism was approximately 10 fold greater when the system was in full operation compared to partial operation.

Similar concentrations of *P. aeruginosa* were observed in most of the dry samples within each sampling event. When the system was not in operation, *P. aeruginosa* was detectable in all of the dry sludge samples at levels similar to those when the plant was in full operation. Values were approximately 10^2 per gram (Table 3). *P. aeruginosa* was not detected in any of the water samples tested.

Only three air samples were positive for fecal coliform (FC) and *Escherichia coli* (EC) bacteria during the sampling events (Table 4). When the system was shut down, only one sample, Air 6, showed the presence of coliform bacteria. Air sample 2 contained EC and FC during both sampling events when the plant was in operation. Levels were 12 and 7 coliform organisms during partial and full operations, respectively (Table 4).

TABLE 1: LEVELS OF AIRBORNE HETEROTROPHIC BACTERIA PRESENT IN THE DEWATERING BUILDING.

Sampling Site		Systems Not in Operation			One Belt Press in Operation			Full Operation		
		TOT[b]	Gm–[c]	Mold	TOT	Gm–	Mold	TOT	Gm–	Mold
Air Samples:										
Staff Office	1RS[a]	375	251	19	65	45	0	1480	992	11
Staff Office	1NRS[d]	281	188	54	59	40	3	821	550	11
Sub CatWalk	2RS	454	304	208	456	306	72	497	333	49
Sub Catwalk	2NRS	478	320	56	400	268	11	1836	1230	16
In Belt Press	3RS	480	321	278	1080	724	45	2732	1830	81
In Belt Press	3NRS	427	286	81	120	1085	8	1296	868	27
Decant House	4RS	389	261	154	1350	905	37	432	289	108
Decant House	4NRS	362	243	19	1350	905	14	313	210	5
Conveyor Belt	5RS	443	297	194	1080	724	38	778	521	32
Conveyor Belt	5NRS	446	299	19	691	463	5	443	297	22
Filter Feed Room	6RS	464	311	62	659	442	9	1188	796	27
Filter Feed Room	6NRS	578	387	16	080	724	14	432	289	11
Vacuum Room	7RS	424	284	46	1004	463	31	1220	817	81
Vacuum Room	7NRS	478	320	78	869	582	14	1512	1013	27
Above Vacuum Filter	8RS		ND[e]			ND		1355	908	86
Above Vacuum Filter	8NRS		ND			ND		2268	1519	43
Non-Sewage Plant	9RS		ND			ND		32	21	3
Non-Sewage Plant	9NRS		ND			ND		16	11	0

[a] RS = Respirable.

[b] Total heterotrophic colony forming units per cubic meter of air.

[c] Gram-negative colony forming units per cubic meter of air.

[d] NRS = Non-Respirable.

[e] ND = Not Determined.

TABLE 2: LEVELS OF HETEROTROPHIC BACTERIA DETECTED IN DEWATERING BUILDING SLUDGE AND WATER.

Sampling Site	Systems Not in Operation	One Belt Press in Operation	Full Operation
Wet Sludge:			
Vacuum Filter 1	1.3×10^{6}[a]	1.8×10^{8}	1.1×10^{7}
Vacuum Filter 2	1.5×10^{6}	9.0×10^{8}	3.1×10^{6}
Vacuum Filter 3	9.4×10^{6}	5.1×10^{7}	2.8×10^{6}
Belt Press 4	ND[b]	3.3×10^{8}	1.4×10^{6}
Belt Press 5	ND	1.1×10^{8}	1.8×10^{5}
Belt Press 6	ND	1.0×10^{8}	5.0×10^{5}
Vacuum Filter 7	ND	ND	7.8×10^{6}
Vacuum Filter 8	ND	ND	6.2×10^{6}
Vacuum Filter 9	ND	ND	3.2×10^{6}
Dry Sludge:			
Vacuum Filter 1	1.6×10^{7}[a]	1.2×10^{8}	1.8×10^{6}
Vacuum Filter 2	3.2×10^{7}	7.0×10^{7}	5.1×10^{6}
Vacuum Filter 3	2.1×10^{7}	1.5×10^{8}	2.7×10^{6}
Belt Press 4	4.0×10^{8}	2.5×10^{8}	2.4×10^{5}
Belt Press 5	3.0×10^{8}	1.7×10^{8}	4.0×10^{5}
Belt Press 6	3.0×10^{8}	1.7×10^{8}	4.0×10^{5}
Floor Sample 7	1.5×10^{8}	4.7×10^{7}	2.0×10^{8}
Water Samples:			
Water 1	5.9×10^{3}[c]	< 1	1.6×10^{4}
Water 2	9.8×10^{4}	8.0×10^{4}	2.2×10^{4}
Water 3	1.2×10^{2}	8.4×10^{5}	7.0×10^{1}
Water 4	6.5×10^{3}	9.3×10^{6}	1.7×10^{3}
Tap Water	ND	< 1	< 1

[a] Colony forming units per gram of sludge.

[b] ND = Not Determined.

[c] Colony forming units per 100 mL of water.

TABLE 3: LEVELS OF *PSEUDOMONAS AERUGINOSA* DETECTED IN THE DEWATERING BUILDING AIR, WATER AND SLUDGE.

Sampling Site	Systems Not in Operation	One Belt Press in Operation	Full Operation
Air Samples:			
1-Staff Office	–[a]	–	–
2-Sub Catwalk	–	–	–
3-In Belt Press	–	–	–
4-Decant House	–	–	–
5-Conveyor Belt	–	–	3[b]
6-Filter Feed Room	–	–	–
7-Vacuum Room	–	–	–
8-Above Vacuum Filter	ND[c]	ND	–
9- Non-Sewage Plant	ND	ND	–
Wet Sludge:			
1-Vacuum Filter	7.0×10^{2}[d]	3.0×10^{1}	1.0×10^{2}
2-Vacuum Filter	7.0×10^{2}	3.0×10^{1}	< 1
3-Vacuum Filter	1.2×10^{2}	5.0×10^{1}	< 1
4-Belt Press	ND	< 1	4.1×10^{2}
5-Belt Press	ND	< 1	2.0×10^{2}
6-Belt Press	ND	< 1	3.0×10^{2}
7-Vacuum Filter	ND	ND	3.0×10^{2}
8-Vacuum Filter	ND	ND	1.0×10^{2}
9-Vacuum Filter	ND	ND	< 1
Dry Sludge:			
1-Vacuum Filter	1.9×10^{2}[d]	1.6×10^{4}	< 1
2-Vacuum Filter	2.9×10^{2}	< 1	< 1
3-Vacuum Filter	7.0×10^{1}	4.0×10^{1}	1.0×10^{2}
4-Belt Press	4.3×10^{2}	1.0×10^{1}	4.1×10^{2}
5-Belt Press	3.2×10^{2}	1.0×10^{1}	5.0×10^{2}
6-Belt Press	4.1×10^{2}	1.0×10^{1}	9.0×10^{2}
7 Floor Sample	3.0×10^{1}	1.6×10^{1}	3.8×10^{2}
Water Samples:			
Water 1	< 1[e]	< 1	< 1
Water 2	< 1	< 1	< 1
Water 3	< 1	< 1	< 1
Water 4	< 1	< 1	< 1
Tap Water	ND	< 1	< 1

[a] Concentration of *P. aeruginosa* per cubic meter of air was < 1 colony forming unit.

[b] Concentration of *P. aeruginosa* per cubic meter of air.

[c] ND = Not Determined.

[d] Concentration of *P. aeruginosa* per gram of sludge.

[e] Level of *P. aeruginosa* was < 1 per 100 mL.

TABLE 4: CONCENTRATION OF FECAL COLIFORMS (FC) AND ESCHERICHIA COLI (EC) IN SAMPLES TAKEN FROM THE DEWATERING BUILDING.

Sampling Site	Systems Not in Operation		One Belt Press in Operation		Full Operation	
	FC	EC	FC	EC	FC	EC
Air Samples:						
1-Staff Office	< 1[a]	< 1	< 1	< 1	< 1	< 1
2-Sub Catwalk	< 1	< 1	12	11	7	7
3-In Belt Press	< 1	< 1	< 1	< 1	< 1	< 1
4-Decant House	< 1	< 1	< 1	< 1	< 1	< 1
5-Conveyor Belt	< 1	< 1	< 1	< 1	< 1	< 1
6-Filter Feed Room	3	3	< 1	< 1	< 1	< 1
7-Vacuum Room	< 1	< 1	< 1	< 1	< 1	< 1
8-Above Vacuum Filter	ND[b]	ND	ND	ND	< 1	< 1
9-Non-Sewage Plant	ND	ND	< 1	< 1	< 1	< 1
Wet Sludge:						
1-Vacuum Filter	1.5×10^{3}[c]	1.5×10^{3}	1.5×10^{4}	1.5×10^{4}	2.0×10^{3}	1.8×10^{3}
2-Vacuum Filter	1.6×10^{3}	1.0×10^{3}	4.4×10^{3}	4.2×10^{3}	1.3×10^{3}	1.0×10^{3}
3-Vacuum Filter	6.5×10^{3}	6.5×10^{3}	1.9×10^{3}	1.9×10^{3}	4.3×10^{2}	4.0×10^{3}
4-Belt Press	ND	ND	1.4×10^{5}	1.4×10^{5}	2.4×10^{3}	2.3×10^{3}
5-Belt Press	ND	ND	9.0×10^{4}	9.0×10^{4}	5.6×10^{3}	4.0×10^{3}
6-Belt Press	ND	ND	1.8×10^{5}	1.8×10^{5}	1.9×10^{3}	1.7×10^{3}
7-Vacuum Filter	ND	ND	ND	ND	8.7×10^{2}	8.5×10^{2}
8-Vacuum Filter	ND	ND	ND	ND	1.4×10^{3}	1.3×10^{3}
9-Vacuum Filter	ND	ND	ND	ND	7.4×10^{2}	7.4×10^{2}
Dry Sludge:						
1-Vacuum Filter	7.8×10^{3}[c]	7.1×10^{3}	2.3×10^{4}	1.6×10^{4}	6.3×10^{2}	6.0×10^{2}
2-Vacuum Filter	7.9×10^{3}	6.0×10^{3}	2.4×10^{4}	2.3×10^{4}	7.1×10^{2}	6.5×10^{2}
3-Vacuum Filter	1.4×10^{4}	1.0×10^{4}	1.6×10^{4}	1.5×10^{4}	8.0×10^{2}	7.6×10^{2}
4-Belt Press	7.4×10^{5}	5.5×10^{5}	1.5×10^{5}	1.5×10^{5}	7.0×10^{3}	7.0×10^{4}
5-Belt Press	4.2×10^{5}	4.1×10^{5}	2.9×10^{5}	2.9×10^{5}	2.8×10^{3}	2.8×10^{3}
6-Belt Press	9.3×10^{5}	7.8×10^{5}	1.2×10^{5}	1.2×10^{5}	1.1×10^{3}	1.0×10^{3}
7-Floor Sample	2.1×10^{2}	1.1×10^{2}	1.6×10^{3}	1.5×10^{3}	7.2×10^{3}	6.4×10^{3}
Water Samples:						
Water 1	< 1[d]	< 1	< 1	< 1	< 1	< 1
Water 2	< 1	< 1	< 1	< 1	4.0×10^{0}	4.0×10^{0}
Water 3	< 1	< 1	< 1	< 1	< 1	< 1
Water 4	< 1	< 1	< 1	< 1	< 1	< 1
Tap Water	ND	ND	< 1	< 1	< 1	< 1

[a] Colony forming units per cubic meter of air.

[b] ND = Not Determined.

[c] Colony forming units per gram of sludge.

[d] Colony forming units per 100 mL sample water.

Wet and dry sludge samples contained coliform organisms ranging from 1.1 x 10^2 cfu/g to 7.8 x 10^5 cfu/g. Approximately 85% of the FC population were EC.

Concentrations of FC or EC present in wet sludge did not seem to vary during the sampling periods. In comparison, the concentration of the coliform population in dry sludge was lower when the system was in full operation. *Escherichia coli* estimates in dry sludge at vacuum filter site 1, for example, ranged from 6.0 x 10^2 EC per gram during full operation to 7.1 x 10^3 EC /g during the shut down operation. EC and FC were detected only once in water sample 2.

Table 5 lists the level of endotoxin that was detected in the dewatering building and adjacent areas during different times of operation. When the system was not in operation, none of the selected air sampling sites were found to contain endotoxin. The maximum level detected over the entire sampling period was 4.2 x 10^{-1} ng/m^3.

When the dewatering building was in partial operation, two air samples, air 2 and 3, were positive for endotoxin. Both of these areas were also found to have an aerosol mist as well as small particles of sludge falling in their immediate vicinity.

When the dewatering building was in full operation, air sampling sites 2, 3, 5 and 8 displayed evidence of endotoxin. The level of endotoxin detected was 0.42 ng/m^3 in each of the areas. An aerosol was also evident at each of these sampling areas.

The endotoxin content of the dry sludge samples was 10 fold greater than the wet sludge samples. Values reached 125 µg/g for wet sludge and 1250 µg/g of dry sludge.

Effluent water samples taken from hoses located in the building were also found to contain endotoxin at approximately 1.25 EU/mL water. In comparison, city tap water contained 0.0125 EU/mL (Table 5).

In total, 22 dry sludge, 18 wet sludge, 23 air samples and 12 water samples were examined for parasitic organisms. The results are listed in Table 6. Free living amoebae were isolated from 63.6% of the dry sludges and 55.6% of the wet sludges. None of the air and water samples yielded parasitic organisms.

Of the 24 isolates of the free-living amoebae only four were *Naegleria* spp. according to the exflagellation and temperature tolerance tests; the remaining twenty isolates belonged to the *Acanthamoeba* group. The *Naegleria* was isolated from the wet sludge only.

Three of the four *Naegleria* isolates and two of the 20 *Acantahamoeba* isolates were pathogenic for the experimental mice, killing the animals in three days to four weeks. Trophoziotes of the amoebae were demonstrated from the brains of the dead mice. No amoebae were detected from brain tissues of those animals which survived for two months after inoculation of the amoebae.

DISCUSSION

Heterotrophic Bacteria

The concentration of airborne bacteria in the dewatering building was found to vary depending on the degree of operation of the equipment. When the system was shut down, the levels of bacteria detected per cubic meter of air averaged 434 cfu. Results showed that a maximum of 67% of the colonies on the plates were determined to be Gram-negative. Therefore, the

TABLE 5: ENDOTOXIN LEVELS IN THE DEWATERING BUILDING.

Sampling Site	Systems Not in Operation	One Belt Press in Operation	Full Operation
Air Samples:			
1-Staff Room	–[a]	–	–
2-Sub Catwalk	–	0.42[a]	0.42
3-In Belt Press	–	0.42	0.42
4-Decant House	–	–	–
5-Conveyor Belt	–	–	0.42
6-Filter Feed Room	–	–	–
7-Vacuum Room Air 7	–	–	–
8-Above Vacuum Filter	ND[b]	ND	0.42
9-Non-Sewage Plant	–	–	–
Wet Sludge:			
1-Vacuum Filter	125[c]	125.0	12.5
2-Vacuum Filter	125	125.0	12.5
3-Vacuum Filter	125	125.0	12.5
4-Belt Press	ND	12.5	12.5
5-Belt Press	ND	125.0	12.5
6-Belt Press	ND	125.0	12.5
7-Vacuum Filter	ND	ND	125.0
8-Vacuum Filter	ND	ND	125.0
9-Vacuum Filter	ND	ND	125.0
Dry Sludge:			
1-Vacuum Filter	1250[c]	125	125.0
2-Vacuum Filter	125	1250	125.0
3-Vacuum Filter	125	1250	125.0
4-Belt Press	125	1250	125.0
5-Belt Press	1250	1250	125.0
6-Belt Press	1250	1250	125.0
7-Floor Sample	125	1250	12.5
Water Samples:			
Water 1	1.25[d]	1.25	12.50
Water 2	1.25	1.25	1.25
Water 3	0.13	0.13	1.25
Water 3	1.25	1.25	1.25
Tap Water	ND	ND	0.01

[a] Endotoxin levels are reported in ng/m^3. A '–' indicates that the sample contained less than 4.2×10^{-1} ng/m^3.

[b] ND = not determined

[c] Micrograms of endotoxin per gram of sludge.

[d] Endotoxin units per mL of water sample (EU/mL).

TABLE 6: PARASITIC ORGANISMS DETECTED IN SAMPLES TAKEN FROM THE DEWATERING BUILDING.

	Number of Parasites Found				
Sample Type	Giardia	Cryptosporidium	Acanthamoeba	Naegleria	Helminths
Air Samples	0	0	0	0	0
Water Samples	0	0	0	0	0
Dry Sludge Samples	0	0	14	0	0
Wet Sludge Samples	0	0	6	4	0

concentration of airborne Gram-negative bacteria was approximately 291 per m^3 air when the systems were shut down (i.e. 434 x 67%). This concentration is considerably lower than the acceptable level of 1000 /m^3 suggested by Rylander and co-workers (1983). Clarke and co-workers (1983) have reported that 1.9×10^4 to 3.7×10^5 Gram-negative airborne bacteria were detected in waste process rooms in compost plants in Sweden.

During partial and full operation, the concentration of airborne bacteria was greater than when the system was not operation. Air sample 3NRS (partial operation) and samples 2NRS, 3RS, and 8NRS (full operation) were of significance because the airborne Gram-negative bacterial levels were calculated to be 1085/m^3, 1230/m^3, 1830/m^3 and 1520/m^3, respectively. Even though these concentrations are above the 1000/m^3 maximum suggested level, only sample 3RS may be of concern. Bacteria in the non-respirable samples 2NRS, 3NRS and 8NRS were associated with particles greater than 5 µm in diameter and would be deposited in the upper respiratory pathway and subsequently be cleared and swallowed into the gastrointestinal tract (Rylander et al, 1983).

The respirable air sample 3RS, taken during full operation, exceeded the 1000/m^3 level. This specific sampling site was located inside a plastic canopy surrounding the belt press. During the sampling events, steam or aerosols were seen to rise over the bed of the belt press. The high level of bacteria detected at this site may be due to the presence of the aerosol particles laden with bacteria. When the building was in partial operation, one other sample was taken from within the canopy over the belt press. During this sampling event, a level of 724 airborne Gram-negative colony forming units was determined.

The concentration of mold colonies detected when the dewatering building was not in operation was greatest as compared to the sampling times when the systems were operating. With elevated airborne mold levels, workers with a sensitivity or allergy to mold may experience some discomfort. However, unless the mold sensitivity of the individual is known, it is difficult to state whether the observed levels were high. In addition, the exact source of the mold is indeterminate in this study as the dewatering building was not a closed system. Outside air was able to enter the building through the open doors and windows, hence the source of molds detected may be environmental. Furthermore, Eikmann and co-workers (1986) were unable to show an increase in air pollution through mold fungi from wastewater treatment plants.

The heterotrophic bacterial level of the sludge samples was calculated to vary from 10^5 to 10^8 cfu/g. These levels are not unexpected. Dry sample 7, taken from the floor, had a high bacterial concentration ($>10^8$/g). It was not known how long the sludge material had been on the floor; however, it was present for the entire sampling period of 1.5 weeks. This suggests that the bacteria are able to survive for certain periods of time in dried sludge.

Effluent water used in washing down the equipment and floors was found to have relatively high levels of bacteria. If pathogenic organisms are present and workers ingest this water, health problems may ensue. Chlorination or superchlorination of this effluent water should greatly decrease the concentration of bacteria detected.

Pseudomonas aeruginosa

Pseudomonas aeruginosa is a resident of the intestinal tract of approximately 10% of healthy individuals (Sonnenwirth, 1980) and in many cases, is regarded as an opportunistic pathogen (APHA, 1985).

Only one of the air samples was found to contain P. aeruginosa over the three sampling periods. Air sample 5, during full operation, yielded 3 colonies of Pseudomonas aeruginosa per m^3 air. The fact that this organism was rarely detected in air may be attributed to its extreme sensitivity to drying (Sonnenwirth, 1980). The air sample 5 location had an aerosol mist which rose over the sludge material and surrounded the sampling apparatus.

Wet and dry sludge samples had relatively low levels of P. aeruginosa. Concentrations varied from < 1 cfu/g to a maximum of 1.6×10^4 cfu/g. Other studies have shown that levels of P. aeruginosa have been as high as 3.7×10^5 per gram of sludge (Harding et al, 1981). In comparison, none of the water samples showed the presence of P. aeruginosa.

Fecal Coliform Bacteria

The level of coliform bacteria present in various samples may give an indication of the extent of sewage contamination or spill into the working environment. Fecal coliform (FC) bacteria were detected in only three of the air samples taken.

The site where air sample two was taken had coliform bacteria detected in both instances when the dewatering building was in operation. Levels were regarded as low since only 12 and 7 FC/m^3 were recovered during partial and full operation, respectively. This particular sampling site, Air 2, was located under a catwalk where sludge droplets and aerosols were evident.

It has been reported that coliform levels have been found to represent between 10 to 50% of standard plate counts (Clarke et al, 1980). Taking this observation into consideration, the total heterotrophic population (Table 2) compared with the coliform levels (Table 5), consists of an extremely small portion of coliform organisms. Results indicate that less than 3% of the total heterotrophic concentration were fecal coliforms.

This same observation was noted with wet and dry sludge samples. The total heterotrophic population concentration was approximately 10^7 /g (Table 2) while the fecal coliform population ranged from 10^2 to 10^5 /g. The concentration of E. coli closely followed the values determined for FC.

Dry sludge sample 7, collected from the floor had a FC population averaging 3×10^3 cfu/g. The length of time that this sample had been on the floor is indeterminate. Fecal coliforms have been reported to survive up to 7 months on dry irrigated soil with a suspension of anaerobically digested sludge (Hyde, 1976). In addition, Hinsley and co-workers (1972) have shown that EC can survive in soil with 5% moisture for one month or longer. Cliver (1980) has also reported that EC is consistently present in sludge of varying ages. The results obtained from Dry site 7 corroborate these findings. Therefore, due to the ability of fecal coliforms to persist in dried sludge for long periods of time, care should be taken when in its handling and disposal.

The plant water used in this building was reported to come from the same source. Only one water sample was positive for the presence of fecal coliforms. Nonetheless, the presence of high bacterial levels (Table 2) and coliform bacteria (Table 4) indicates that this water should not be used for human consumption.

Endotoxin Levels

In view of the apparent importance of bacterial endotoxin as a causative agent in occupational disease following inhalation of bacteria-containing dusts, high levels of endotoxin in a working environment could lead to health disturbances.

In this study, airborne endotoxin was detected during partial and full operation. During partial operation, air sampling sites 2 and 3 were found to have 0.42 ng endotoxin $/m^3$ air (Table 5). The same level was detected during full operation at sites 2, 3, 5 and 8 (Table 5). At each of these sites, an aerosol was visible in the vicinity of the sampling machinery. Since bacceria may be present in aerosols generated around sludge processing equipment it is expected that endotoxin may also be present. In addition, endotoxin may be present in the absence of viable bacterial cells.

The level of endotoxin detected was below the 100 ng/m^3 air level suggested by Rylander and coworkers (1983). Therefore, the levels detected in the dewatering building are well within the proposed allowable limits.

The concentration of endotoxin in sludge was considerably greater than that detected in air. Values ranged from 12.5 to 1250 µg/endotoxin per gram of sludge. It is reasonable to assume that aerosolization of the sewage sludge and consequently endotoxin could be made possible by its association with fine wash spray, rising steam from the heated sludge or generation of airborne dust particles by walking on or disturbance of dried sludge. However, in this study, the levels of airborne endotoxin generated was below suggested maximum allowable limits.

Endotoxin was also detected in each of the water samples tested. Compared to city tap water, effluent water contained 10 to 1000 times greater concentrations of endotoxin.

Parasites

In view of the wide distribution of free-living amoebae in nature, it is not surprising to isolate them in a large number of sludge samples. It is interesting however that *Naegleria* was not isolated from the dry sludges. Although *Naegleria* has been isolated from the nasal cavity of persons who had not been swimming (Chang, 1971), suggesting that viable cysts may be airborne, to date all *Naegleria* isolates were associated with water.

The isolation of pathogenic *Naegleria* in the present survey supports the previous reports that virulent species of *Naegleria* do occur in Canada, as demonstrated in Ontario lake waters (Health and Welfare, 1980), sewage (Seyfried *et al*, 1984) and soils (Scholten, 1979).

Acanthamoebae were isolated from 50% of 40 sludge samples whereas *Naegleria* was isolated only from 10% of the sludge. It is well known that cysts of *Acanthamoeba* can withstand dessication, freezing and a variety of antimicrobial agents. The cysts are also resistant to levels of chlorine that are routinely used to disinfect drinking water and swimming pools.

Although a majority of *Acanthamoeba* isolates in this study were nonpathogenic for mice, the number of organisms in the inoculum, among other factors, may be responsible for the mouse survival. It is also possible that even though some of the isolates were avirulent for mice, they may cause keratoconjunctivitis in man.

None of the test samples revealed intestinal helminths and protozoa of medical importance, suggesting that intestinal parasites are not a major problem in sewage treatment.

SUMMARY

An extensive examination of the bacterial and parasite levels in the dewatering building was performed during different stages of operation.

Results showed that greater levels of airborne bacteria and endotoxin were present when the building was operating; however, only one sampling site, an enclosed canopy surrounding an operating belt press, had greater than the suggested level of 1000 Gram-negative bacteria per cubic meter of air. Plant water used to wash down machinery was found to have varying levels of endotoxin and Gram-negative bacteria and therefore should not be used for human consumption. Parasites were associated with wet and dry sludge samples only. Human exposure should be minimized when handling sewage sludge through the use of gloves, masks and proper clothing. It is proposed that a full epidemiological study be run in conjunction with bacterial, virus, endotoxin and parasite sampling to assess specific agents that may affect the workers.

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INHIBITION KINETICS MODELS FOR THE BIOLOGICAL TREATMENT OF WASTEWATER

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ABSTRACT

At present time, there area over 6500 chemical products of common use. These materials are quite often discharged directly into a receiving body of water with little or no treatment at all. As a consequence of this and due to the ever increasing rate of discharge of these chemicals, several water courses are being threatened and their autopurification capacity is nearly exhausted.

Wastewater biological treatment processes have made a notable impact in that biodegradable compounds may be effectively removed from an industrial discharge. However, there are many compounds which are refractory under normal wastewater treatment plant operating conditions and not amenable to biodegradation.

These refractory products leave the treatment units unchanged, at the same concentrations as they entered, and on occasions they may even inhibit the micrograms' metabolism. These chemicals are a cause of real concern; they threaten human and animal health and may percolate through the ground reaching groundwater supplies.

Several authors have discussed the biochemical aspects of biodegradability, toxicity and stabilization of some of these toxic compounds. This work deals with the operational aspects of a conventional activated sludge treatment plant with a hydraulic regime corresponding to piston flow with solids recirculation. The system being considered is fed with at least one biodegradable compound which acts as an inhibitor of the metabolism of the microorganisms.

The degree of biodegradability is given by the utilization constant of the inhibitor as a secondary substrate. The effect of the presence of the inhibitor on the process is determined principally by the way in which the inhibitor acts over the enzime (ie the microorganisms) which degrades the

organic substrate.

The analysis was done using mathematical models based on Monod's and Novick-Sziland's theories and on three reversible mechanisms of enzimatic inhibition based on Michaelis-Menten enzimatic kinetics.

The models developed allow to predict the behaviour of the system in the presence of different types of inhibitors characterized by Monod's apparent constant and by the maximum apparent rate of the reaction between the enzime and its substrate (which depend on the formation constants of the enzime-inhibitor complexes and/or enzime-substrate-inhibitor complexes). Also, it is possible to design control strategies to compensate for the decrease in efficiency of the process.

SUPERCRITICAL WATER AND SOLVENT OXIDATION

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ABSTRACT

Alternatives for treatment and disposal of toxic wastewaters and large volumes of sludges are urgently needed. The amount of toxic wastes and wastewater treatment plant sludges produced each year is enormous and the volume continues to grow. To resolve this dilemma, it is necessary to research, develop, design, and utilize more cost-effective and efficient treatment systems. Future levels of treatment are generally beyond present capabilities of secondary (biological), tertiary (physical-chemical), and destruction (incineration) capabilities. To achieve reasonable cost limits, the ultimate goal of waste management must involve new technologies. As such appropriate use of subcritical and supercritical fluid reactors offers considerable promise for ultimate treatment of special wastes.

Background information is provided on the magnitude of the current waste problem, wet air oxidation, and the applicability of subcritical and supercritical water oxidation methodologies. Emphasis is directed to a discussion of: (a) deep-well, subcritical, full-scale 90.7 MT per day (100 dry ton) municipal sludge facility at Sims Bayou, Houston, Texas, USA; and (b) the extensive supercritical research activity at The University of Texas at Austin.

INTRODUCTION

In the development of "third round" National Pollution Discharge Emission Standards (NPDES) permits, EPA has moved beyond technology-based controls and water quality evaluations as based on conventional parameters. The focus on the new "Post BAT" permits will be to ensure that adequate controls are being implemented to confirm the protection of human health and aquatic life on a site-specific receiving stream basis. No source will be allowed to discharge any wastewater which will result in the endangerment of a drinking water supply, aquatic bioaccumulation which threatens human health, instream acute or chronic aquatic toxicity, or a violation of an applicable general or numerical state water quality standard.

In 1982, the wastewaters from domestic and industrial sources, respectively, produced sludges in the amount of 17.8 million metric tons (MMT) and 16.2 MMT (19.6 and 17.9 million tons) of sludge per year, dry weight basis. To comply with future waste management practices, the volume of this sludge must be reduced. Unless new technologies are used, the cost of sludge volume reduction will be exceedingly high. Even today the cost of treatment and handling sludges is about 50% of wastewater treatment.

Hazardous wastes generated annually by industry continue to grow. (1) Industrial contributions (USA) total over 266 million metric tons (MMT), and by 1990 the U.S. Environmental Protection Agency (EPA) estimates this level will increase to 280 MMT. Chemicals and allied products account for 30% of this total. There are about 70,000 and 125,000, respectively, large and small hazardous waste generators. A small generator is classified as one which produces between 100 and 1,000 kg of hazardous wastes per month

The development of a system to accommodate the regulation, operation, and management of the hazardous waste generators will be exceedingly difficult. Due to an estimated decrease in the total available land disposal facilities of 68% (516 from 1,635), the average waste per facility per day will increase by 2,500 MT or about 240%.

According to a survey conducted by the Association of State and Territorial Solid Waste Management Officials, about 30,000 suspected hazardous waste sites have been identified in the U.S. Of these, 7,9[illegible]0 are confirmed, but 90% are not on the National Priority List for federal funding.

Under the 1986 Superfund amendments and Reauthorization Act (SARA) the concept that "the polluter should pay" has developed a new dimension. Now the federal government can use two powerful principles of liability: Joint and Several Liability and Strict Liability. In the former, parties responsible at a Superfund site can be sued together or separately for 100% of the clearnup costs. Strict liability is liability without a showing of fault.

WASTEWATER TREATMENT, DETOXIFICATION, AND DISPOSAL OPTIONS

Wastewaters containing organic materials represent the largest fraction of toxic and hazardous wastes produced. All of these wastewaters must be treated and detoxified prior to release to the environment. Also, as the volumes of municipal and industrial wastewaters are increased and the levels of treatment escalated, greater amounts of sludges must be conditioned and managed.

Historically, there have been alternatives for the disposal of residual wastewaters and sludges. Under today's environmental constraints, many of these options are not viable. These options have been summarized in Table 1. The magnitude of the waste treatment and disposal problem, required efficiencies, economic considerations, and growing public involvement mandate a reevaluation of existing treatment and disposal concepts.

Today, when wastes are not acceptable for biological treatment, the alternative method of disposal is hauling as a hazardous waste under RCRA regulations. Reported charges (probably low) for hauling hazardous wastes are as follows: (2)

- Barrelled waste: $50 to $70 per 55 gal drum plus $3 per loaded mile shipping ($0.24 to $0.34/L plus $2.80/km)
- Bulk waste: $0.55 to $0.75/gal plus $3 per loaded mile shipping ($0.15 to $0.20/L plus $2.80/km).

As a comparison, municipal wastewater treatment costs and secondary biological treatment costs are about $0.13/1,000L ($0.50/1,000 gal). In addition, municipalities generally charge industry $0.01 to $0.02/kg ($0.05 to $0.10/lb) of COD removed. The investor's interest rate of return (IIR) for the wet air oxidation (WAO) process would require a charge of $0.2/kg ($0.16/lb) of COD removal (20% IIR).

For materials such as 2,3,7,8-tetrachlorodibenzoidioxin, even total incineration, temperature of 1,600 °C, presents a challenge. (3) The costs are high.

Table 1. Alternatives for Treatment and Disposal of Wastewaters

	Technique	Acceptability
1.	Dilution	Not permissible.
2.	Biological/Physical/Chemical	Economical, efficient for dilute biodegradable wastes (≈50% to 90⁺% BOD and COD removals)
3.	Deep-Well Injection	Costly, technically feasible, but progressively unacceptable.
4.	Solar Evaporation	Increasingly unacceptable because of potential air and groundwater pollution.
5.	Land Disposal	Potential groundwater pollution and crop contamination.
6.	Oxidation Regeneration	Usually applicable for waste concentrations below 1%, but not widely acceptable.
7.	Incineration	Expensive but adaptable for concentrated wastes (>20%); subject to air pollution regulations; requires temperatures of about 900 ^{o}C to 1100 ^{o}C and residence time of 2 sec. Stack gases and ash generally questioned by public.
8.	Ozonation	Limited by rate of mass transfer, produces residual low-molecular weight compounds which are oxidized slowly.
9.	Wet Air Oxidation (WAO)	Applicable for low pressure sludge conditioning, not applicable for destruction. When operated as a low-pressure sludge conditioner (50% to 90% removal).
10.	Subcritical (Deep-Well)	COD reduction about 70% to 80%.
11.	Supercritical (Deep-Well)	Destruction of all toxic wastes and sludges; enhanced efficiency and reduction in capital cost; applicable for destruction of organic compounds; efficient counter current heat exchanger; short reaction times; and suitable for large flows.

BACKGROUND--WET AIR OXIDATION (WAO)

Extensive literature is available describing WAO. (4,5,6) The WAO concept as typified by some of the patents that have been issued:

Bayer	1888	Bauer	1969
Strehlement	1912	Lawless	1971
Proteus	1954	Burleson	1986
Zimmerman		Titmas	1986

One of the first WAO processes for sludge treatment was based on the Zimmerman Patent. The Zimpro process (Zimpro, Inc., a subsidiary of Sterling Drug Inc.) utilized relatively low temperatures and pressures to condition sludges.

WAO occurs through a family of related oxidation and hydrolysis reactions: temperatures of 175 °C to 320 °C (347 °F to 608 °F) and pressures of 2,169 kPa to 20,708 kPa (300 psig to 3,000 psig). Oxygen may be provided by either compresed air or a high pressure oxygen system. Theoretically, WAO can proceed autogenously (self-sustaining) with chemical oxygen demand (COD) of about 15,000 mg/L as compared to 300,000 mg/L to 400,000 mg/L for incineration. Under ideal conditions sulfur is oxidized to sulfate and remains in the aqueous phase. Organic nitrogen is converted primarily to NH_3, $NOSO_x$ or NO_x. Halogens remain in the aqueous phase. Metals generally remain in their aqueous phase (dissolved or suspended) after being converted to the highest oxidation state. The discharged gases under ideal conditions are primarily spent air and CO_2.

Examples of bench-scale and full-scale WAO results, respectively, are shown in Tables 2 and 3 (6). Catalysts such as nitrate and bromine anions may facilitate higher organic destruction rates of organic chemicals at lower temperatures (165 °C to 250 °C) as compared to conventional wet oxidation. (7)

In batch ozonation and WAO experiments using phenol, 2-chlorophenol, 1,2 dichloroethane and dimethylphthalate, acetic acid was not evident during ozonation, although acetic acid was a major end product of wet oxidation. (8) The reverse was true for oxalic acid. Ozonation destroyed all four pollutants in 5 to 10 minutes. The removal rate was limited by the rate of ozone mass transfer. Ozone utilization ratios indicated that 3.9 to 5.9 moles of ozone per mole of specific pollutant were consumed. Tests conducted at 232 °C indicated that a significant amount of TOC remained in solution after the specific pollutant had been destroyed and the remaining TOC (mostly low-molecular weight acids, acetaldehyde and acetone) was slowly oxidized.

Since the cost of treatment and disposal of sludges produced by biological treatment plants represents about 50% of present secondary treatment costs, WAO provides some exciting possibilities. At about 14.5 MPa (145 bar) and temperatures of greater than 275 °C WAO treatment achieves about 86% volatile acid reduction. (9) Under WAO conditions, insoluble organic matter is converted to short chain organic acids (mostly acetic), ammonia, nitrate, nitrite, organic nitrogen, and carbon dioxide. Also, settling characteristics of the solids are improved. Some WAO processes have exhibited unusually high construction and operation costs when treating municipal sludges. In this case the pressure was about at 10 MPa (100 bar).

SUPERCRITICAL FLUIDS

Much has been written about supercritical fluids (SCF), supercritical extraction, and supercritical water oxidation (SCWO). (10, 11, 12, 13) Special SCF are too attenated to be true liquids and too dense to be true gasses. SCF occur when both tempertures (T_C) and pressures (P_C) within the fluid exceed the critical point. The relationships among solid, liquid, gas, and supercritical state of water are shown in Figure 1. In this figure, the critical point of water is defined by its critical temperature and critical pressure which are 374 °C (705.5 °F) and 218 atm (3,206 psia) respectively. Under supercritical conditions, the density, dielectric constant, hydrogen bonding, and other physical properties of water are altered such that it behaves as a moderately nonpolar liquid. (14) For example, benzene becomes miscible in all proportions in water under SCF conditions. Supercritical water can also liquefy and degrade polymeric materials, such as wood. However, under the same conditions the solubility of NaCl and CaCl, respectively, might be as low as 100 mg/L and 10 mg/L.

Table 2. Bench-Scale WAO of Pure Compounds (6)

Compound	Conditions (°C/min.)	Starting Conc. (mg/L)	Final Conc. (mg/L)	Destroyed (%)
Acenaphthene	275/60	7000	0.5	99.99
Acrolein	275/60	8410	<3	>99.96
Acrylonitrile	275/60	8060	80	99.0
2-Chlorophenol	*275/60	12410	15	99.88
2,4-Dimethylphenol	275/60	8220	0.1	99.99
2,4-Dinitrotoluene	275/60	10000	26	99.74
1,2-Diphenylhydrazine	275/60	5000	6	99.88
4-Nitrophenol	275/60	10000	40	99.6
Pentachlorophenol	*275/60	5000	135	97.3
Phenol	275/60	10000	20	99.8
Formic Acid	300/60	25000	410	98.3
Chloroform	275/60	4450	3	99.9
Carbon Tetrachloride	275/60	4330	12	99.7
1,2-Dichloroethane	275/60	6280	13	99.8
N-Nitrosodimethylamine	275/60	5030	22	99.6
Hexachlorocyclopentadiene	300/60	10000	<15	>99.9
Toluene	275/60	4330	12	99.7
Nitrobenzene	*320/120	5125	255	95.0
Chlorobenzene	*275/60	5535	1550	72.0
Pyridine	*320/120	3910	570	85.4
2,4-Dichloroaniline	*275/60	259	<0.5	>99.8
2,4,6-Trichloroaniline	*320/120	10000	2.5	99.97
Dibutylphthalate	275/60	5230	26	99.5
Isophorone	275/60	4650	29	99.4
1-Chloronaphthalene	*275/60	5970	5	99.92
Pyrene	275/60	500	0.26	99.95
Malathion	250/60	11800	18	99.85
Kepone	*280/60	1000	690	31.0
Arochlor 1254	320/120	20000	7400	63.0
1,2-Dichlorobenzene	*320/60	6530	2017	69.1

*Catalyzed

The changes in the characteristics of water as it is subjected to supercritical conditions are shown in Fig. 2. At elevated temperatures and pressures a fluid has much greater ability to act as a solvent because the internal energy of the SCF is sufficient to overcome the internal bonding of the fluid molecules (hydrogen bonding disappears in the case of water). The great solvent properties of SCF at low density provide high diffusivity and rapid mass transfer. (15)

Table 3. Full-Scale, WAO of Hazardous Organics (6)
(Deitrich, M.J., 1985)

Study	Waste	Compound/ Parameter	Conditions (^{o}C/min.)	Feed Conc. (mg/L)	Conc. (mg/L)	Effluent Removal (%)
A	Herbicide	Herbicide by product	245/60	735	<5-13.3	98.2->99.3
B	Spent Caustic					
		COD	268/113	108,100	11,600	>89.3
		Total Phenols		15,510	36	99.8
		Organosulfur		3,010	180	94.0
C	Rocket Fuel	COD	271/117	42,130	2,540	94.0
		Hydrazine		12,700	<0.1	>99.99
		1,1-Dimethyl hydrazone		9,250	<0.7	>99.99
D	Pesticide	COD	281/182	110,000	5,200	95.3
		DOC		26,600	1,070	96.0
		Dinoseb		37.1	0.186	99.5
		Methoxychlor		8.84	<0.018	>99.8
		Carbaryl		30.0	0.59	98.0
		Malathion		93.1	0.13	99.9

Generalized Relationships

For a generalized organic substance such as $C_iH_jO_kN_lS_mCl_nI_p$, in a liquid phase, the stoichiometric coefficients, v_1 . . . v_8, for associated substances can be represented by Table 4. Theoretically, the oxidation reaction will require v_2/v_1 moles of oxygen. Less than stoichiometric amounts of oxygen will establish conditions favorable for the creation of CO.

Supercritical Water Oxidation

The SCWO process embraces the oxidation of organics, with air or oxygen in the presence of high concentrations of water under temperatures and pressures above the c.p. values of water, 374 ^{o}C (705.5 ^{o}F) and 22,120 kPa (3,206 psia, 22.12 bar or 218 atm). (14) The C.P. lies on the vapor-liquid saturation dome, 374 ^{o}C and density of 0.1 to 0.5 g/cm^3, a dielectric constant of 3 to 10, and nearly 100% solubility of organics The surface tension of water under these conditions becomes zero. The thermal energy contained in the water molecules reduces hydrogen bonding, and therefore SCW becomes an excellent solvent for nonpolar organic substances. A position above the C.P. could be used to modify concentrated organic liquids, vapors

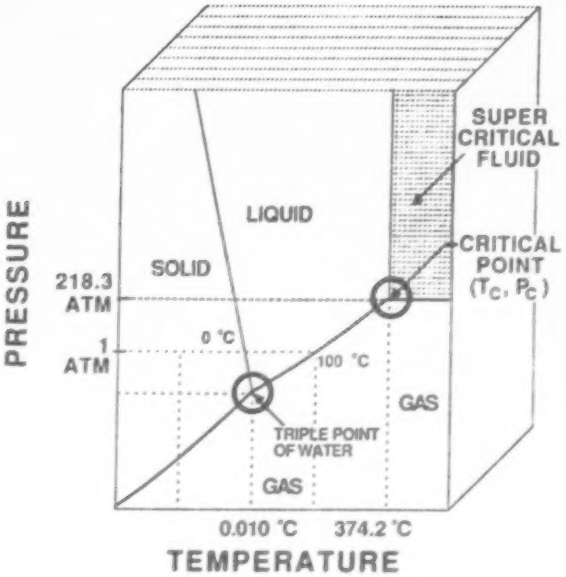
SUPER CRITICAL FLUID
LIQUID
SOLID
CRITICAL POINT (T_C, P_C)
PRESSURE
218.3 ATM
1 ATM
0 °C
100 °C
GAS
TRIPLE POINT OF WATER
GAS
0.010 °C
374.2 °C
TEMPERATURE

Figure 1. Phases of Water at Moderate Pressures (14)

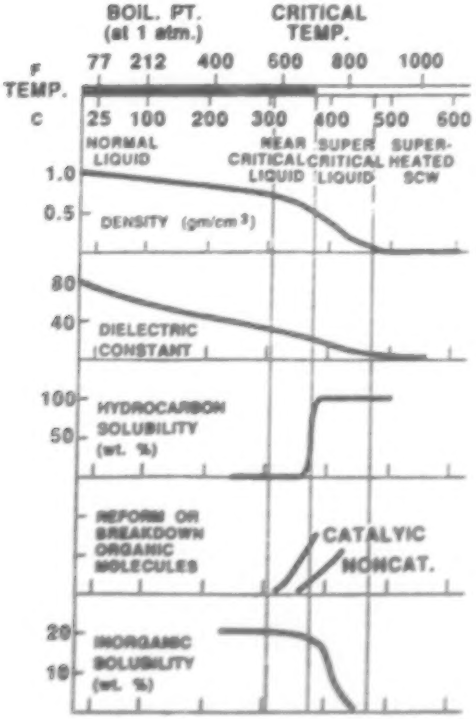

Figure 2. Properties of Water (218.3 - 300 atm.) (14)

Table 4. Atom Balance for the Oxidation of the Species (16)

Atom	Species	Reactants	Products
C	carbon	i	v_3
H	hydrogen	j	$2\ v_4 + v$
O	oxygen	$k + 2\ v_2$	$2\ v_3 + v_4 + 2\ v_6$
N	nitrogen	l	$2\ v_5$
S	sulfur	m	v_6
Cl	chlorine	n	v_7
I	inert	p	v_8

Stoichiometric Coefficients

$v_1 = -1$

$v_2 = (-1/2)\ [2i + (j - n)/2 + 2m - k]$

$v_3 = i$

$v_4 = (j - n)/2$

$v_5 = 1/2$

$v_6 = m$

$v_7 = n$

$v_8 = p$

Chemical Reaction

$$0 = v_1\ C_iH_jO_kN_lS_mCl_nI_p + v_2\ O_2 + v_3\ CO_2 + v_4\ H_2O) + v5\ N2 + v6\ SO_2 + v_7\ HCl + v_8\ I$$

and solids, aqueous solutions containing dissolved or suspended organics, or inorganic solids and adsorbed organics.

The conversion of organic materials in a waste to CO_2, H_2O, and acids or oxyacids of heteroatoms and metals under SCWO conditions is a function of the reaction temperature and residence time. The products of organic nitrogen and ammonia, below 650 ^oC, are primarily N2 and N_2O. The latter decomposes to N_2 and O_2. SCWO at about 400 ^oC (750 ^oF) does not destroy ammonia. Heteroatoms (chlorine, sulfur, and phosphorus) are oxidized to the respective acid forms or corresponding salts if sufficient cations are present.

The SCWO process provides a very strong oxidizing environment It is expected that 80% of the metals in a waste stream are likely to be incorporated in the ash. Calcium sulfate is expected to form. Calcium passing the reactor may be divided between ash and liquid. Sulfates may be split between ash and liquid. The carbonate and sodium at effluent pH of 4.0 is expected to be soluble.

Under SCWO conditions·(374 ^oC to 450 ^oC) salts may remain dissolved or condense as a concentrated brine solution or as a solid particulate (>450 ^oC). The heavy metals may form carbonates or oxides and depending on their volatility may precipitate. The organics are converted to CO_2 and H_2O during the oxidation stage. Char, organic solids, which will not dissolve in supercritical water, will most likely oxidize slowly.

At high temperatures, organics with steam (>800 ^oC) are reformed to CO and H_2. At lesser temperatures (350 ^oC to 450 ^oC), naphtha and other light hydrocarbons react with steam to form, predominantly, CH_4 and CO_2. At 300

°C and above, glucose reacts rapidly and forms a char at a rate of formation at increasing temperatures. However, according to the Modell, et al. (1978) Patent No. 4,113,446 at supercritical water conditions the end products were entirely CO_2, CH_4, and light hydrocarbons.

The gas-to-liquid mass transfer limitation is eliminated at SCWO conditions. This is an important aspect of SCWO, because at even 315 °C (600 °F) and 13.79 MPa (2,000 psi) the solubility of oxygen is still only 0.56% by weight. (16) Oxygen, under SCWO, mixes completely with water and wastewater mixtures, and with a zero surface tension oxygen can penetrate minute pores. Thus, under SCWO conditions organic materials can be oxidized to CO_2 and water very rapidly (less than 1 minute).

Since the reaction occurs entirely in the liquid phase with the heat of combustion being released to the pressured liquid in the reaction zone, WAO differs from conventional oxidation. The released heat can be estimated as follows:

$$C + O_2 = CO_2 + 14{,}100 \text{ Btu/lb C } (32{,}800 \text{ kJ/kg C})$$

$$2H_2 + O_2 = 2H_2O + 61{,}000 \text{ Btu/lb } H_2 \ (141{,}900 \text{ kJ/kg } H_2)$$

Supercritical CO_2 EPA has constructed a pilot-plant at its Research Triangle Park operations to evaluate CO_2 as a desorption medium for removing pesticides from granular activated carbon prior to subsequent regeneration of the carbon. (14) CO_2, unlike water, exhibits a C.P. at a relatively lower temperature and pressure. Supercritical CO_2 appears to be a likely candidate for extraction and disposal or reuse of ketones, aldehydes, and some esters. It has limited propensity to dissolve aromatics, especially biphenyls and naphthalenes and most phenols. Supercritical CO_2 has been used to extract polychlorinated biphenyls (PCBs), 2,2-bis(p-chlorophenyl)-1,1,1-trichloroethane (DDT), and toxaphene from soils. Extractions have varied from 70% to 90%. (18) The use of entrainers such as methanol may increase the efficiency from 70% for DDT removal to 95%. (19)

DEEP-WELL WATER OXIDATION

Subcritical deep-well reactor research began in 1975. EPA initiated the Vertical Tube Reactor Pilot Plant Project at Lowry Air Force Base, Colorado.) This reactor, 450 m (1,500 ft) deep and 6.35 cm (2.5 in) diameter, processed municipal sludge at a flow rate of 19 l/min (5 gpm) and operating temperature of 260 °C (500 °F).

Longmont Experience

A full-scale, deep-well, subcritical reactor was constructed in May 1984 at Longmont, Colorado: 25.40 cm (10 in) diameter; depth of 1,500 m (5,000 ft); and temperature of 300 °C (575 °F); and bottom hole pressure of 10.35 MPa to 13.8 MPa (1,500 to 2,000 psia). (21, 22) For over 1.5 years the Longmont reactor processed 4.5 to 18 MT (5 to 20 T) of municipal sludge 24 hrs/day and 7 days/week. Organic destruction was 80% or greater. The ash passed EPA toxicity tests.

Originally the facility was designed to use compressed air to handle Longmont's 5,500 kg/day of sludge. However, in November 1984 the oxygen source was changed to liquid oxygen which tripled the capacity and enhanced the operating range. The Longmont experience led to advanced subcritical reactor designs (Oxidyne & Vertex) and increased interest in supercritical operations. (Oxidyne & Modar)

EPA goals for hazardous waste management include destruction levels to 99.9999% in a "Totally Enclosed Treatment Facility" (TETF). Bench-scale SCWO results indicate that these goals are achievable. Three systems are briefly described as follows:

The Oxidyne Inc. Process

One of the major differences between surface and deep-well SCWO systems is the regenerative heat exchanger. In the deep-well system, it is possible to take advantage of the insulating effect of the surroundings. In a surface SCWO system the heat exchanger design might require a tube and shell unit with low shell length to diameter ratio in order to minimize shell surface area. The object is to minimize heat loss. As a result of the differences in heat exchanger requirements and other differences, the deep-well system reportedly requires less surface space.

The deep-well concept proposed by Oxidyne Inc. is an example of a process utilizing oil field technology to achieve desired subcritical or supercritical temperature-pressure conditions. The basic differences in subcritical and supercritical designs are presented in Table 5. Various patents support the Oxidyne concepts. These are listed as follows:

1. Patent, 3,853,759, Titmas, Dynamic Hydraulic Column Activation Method.
2. Patent, 3,606,999, Lawless, Method of and Apparatus for Carrying out a Chemical or Physical Process.
3. Patent, 3,449,247, Bauer, Process for Wet Oxidation of Combustible Waste Materials.

Oxidyne Inc. has developed proprietary computer simulation models which calculate heat balances and model the hydrodynamic parameters within the reactor. Heat losses can be estimated based on operating temperatures, flow rates, reactor length, casing thickness, operating regimes, and variability in rock formations. A heat balance model provides the required information for estimating heating, reaction, and cooling operations Hydrodynamic programs calculate downhole pressures, velocities, pressure losses, two-phase flow dynamics, and reaction/residence times.

The Sims Bayou Facility

The Oxidyne Sludge Processing Facility (SPF) is a subcritical unit. It consists of a municipal sludge pretreatment and equalization unit, a deep-well reactor, post reactor gas/liquid separation, ash-liquid separation, cooling, anaerobic polishing unit, off-gas destruction, ash dewatering and disposal, and appurtenances. The depth will be 1,556 m (5,100 ft) with pressures ranging from 13.8 kPa to 13.8 MPa (20 to 2,000 psig). The temperatures will range from ambient to 329 °C (625 °F). The SPF is located on about one hectare (2.3 acres) of land. It is designed to process 90.7 MT (100 ton dry wt) of sludge. The influent solids concentration will range from about 1% to 3.5%. The SPF is expected to provide removals as follows: TSS = 95%, VSS = 98%; COD = 95% and BOD = 93%.

SURFACE WATER OXIDATION REACTOR

The MODAR Oxidation Process is an example of a surface SCWO process. Bench and laboratory-scale units have reached destruction efficiencies of

99.99% to 99.999%. The MODAR unit is also a completely enclosed system. (24, 25,26) Classes of compounds that have been tested are: aliphatic hydrocarbons (Cyclohexane), halogenated aliphatics (1,1,1-trichloethane), aromatic hydrocarbons (o-Xylene), halogenated aromatics (PCB 1254), ketones (methyl ethyl ketone), and organic nitrogen compounds (2,4-dinitrotoluene).

Table 5. Comparisons of Subcritical & Supercritical, Deep-Well Reactor Designs (23)

Parameter		Subcritical	Supercritical
Well depth	- m	1200-1700	2400-3058
	- ft	(4000-5500)	(8000-12000)
Temperature	^{o}C	260-320	400-510
	(^{o}F)	(500-600)	(750-950)
Pressure	KN/m^2	10000-14000	22000-31000
	psia	(1500-2300)	(3200-4500)
Detention time (min)		30-40	0.1-2.0
COD removal (%)		80-85	100
Oxidant		oxygen gas	oxygen gas
Injection point		lower 1/2	lower 1/20
Velocity	m/s	1.2-2.7	1.5-4.6
	(ft/s)	(4-9)	(5-15)
Applicability		sludges, organic wastes, some hazardous wastes	all organic wastes; all hazardous wastes
Waste Destruction Efficiency		depends on organic species range: 80-99.9%	99.99+ %

Bench-scale SCWO reactor results show fuel self-sufficiency when the organic material is greater than 2% in the wastewater. When the organic content is 10% or more, power recovery is feasible.

The reported absence of corrosion is due to the low dielectric constant of water and the absence of ionized species. However, cavitation erosion has been observed in areas where the reactor effluent cooled into the subcritical region. Chloride and probably oxygen levels contributed to corrosion/erosion.(25)

UNIVERSITY OF TEXAS EXPERIMENTAL FACILITY

A number of faculty and graduate students are engaged in a variety of supercritical experiments at UT-Austin. While the studies involving wastewaters and sludges are just beginning, several years of SCF studies have provided considerable background data and experience.

Facilities

The engineering component of the SCF and SCWO studies is located in a new $5 million Separations facility. This unit contains a SCF pilot plant. This 100 mm (4-in) high pressure system is used extensively for extraction purposes. Two laboratory, continuous flow SCWO reactor systems will be completed and installed by December. One of these units will have a capacity of 10 to 50 ml/min aqueous feed, up to 10% total solids; maximum chemical oxygen demand 60,000 mg/l; maximum heat value 1,163 kJ/kg (500 Btu/lb); pressure range 6.895 MPa to 27,580 MPa (1,000 to 4,000 psia); maximum temperature 500 ^{o}C (932 ^{o}F); residence time 0.5 to 5 min; and utilization of either pure oxygen or enriched air with flow rates equivalent to 50% excess of stoichiometric requirements.

Another unit, somewhat smaller, will be designed to utilize H_2O_2 in lieu of O_2. Bench-scale batch reactors are available at both the Chemical Engineering Department laboratories and the Separations facility.

Solvent Extraction

Dense gaseous or supercritical fluid carbon dioxide has several characteristics that are desirable for extractions and purifications. It is non-toxic, inexpensive, and may be used at mild temperatures since the critical temperature is 31 C. Supercritical solvents are unique in that the density and likewise the solvent strength are sensitive functions of pressure and temperature in the highly compressible near critical region. Complex mixtures have been fractionated with high selectivity since there is a great deal of flexibility in adjusting the solvent strength.

In supercritical extraction processes, the economics are influenced strongly by the solvent to feed ratio, which can vary a great deal among different applications, due to the wide range of solubilities. In pure supercritical fluid solvents, solubilities are related primarily to the vapor pressure, and only secondarily to solute-solvent interactions in the fluid phase (27). Differences in vapor pressure are sufficient in many applications to provide for highly selective separations. For separations of species of similar volatility, the use of co-solvents can improve selectivities significantly in systems that form hydrogen bonds (28).

A series of sterols including cholesterol, stigmasterol, and ergosterol as well as pregnenolone, and cholestenone were studied quantitatively to provide a basic understanding of solubilities of complex molecules in carbon dioxide. The key was to determine the vapor pressures which were unavailable. The vapor pressures, which are extremely small and in the range of 4×10^{-6} to 10^{-4} Pa (10^{-5} Pa = 1 bar), were measured using a flow technique along with capillary gas chromatography. In pure CO_2, it was shown that the enhancement factor, or y_2P/P_2^{sat}, where y_2 is the mole fraction of solute, is about the same for each sterol at a given temperature and pressure. This means that the differences in the solubilities are due to the differences in vapor pressures, and that the effect of intermolecular interactions in the supercritical phase is only minor. The selectivity of CO_2 for certain sterols can be quite high because of the large difference in vapor pressures, which is due to packing effects in the solid phase.

The selectivity, that is the solubility ratio of the two sterols in carbon dioxide, is equal approximately to the ratio of the vapor pressures. Acetone increases the solubilities of the two components to a large degree, although it does not decrease the selectivity significantly. The cohesive energy density, or likewise the solubility parameter, of pure carbon dioxide is too low to make it a good solvent for sterols, while that of methanol or ethanol is too high. For this reason, a blend of carbon dioxide and ethanol is a better solvent than either component alone. Liquid decanol is an excellent solvent since the solubility parameter is similar to that of cholesterol. This is an important point as it has been thought that the best liquid solvent for a solute would also be the best co-solvent to add to carbon dioxide. The solubility would be increased more by the addition of several mole percent ethanol than decanol, since the solubility parameter of the blended solvent is still too low in the latter case. This type of behavior is much more complex for the cholesterol-stigmasterol system since co-solvents can form hydrogen bonded complexes in the solid phase, which can lead to large improvement in the selectivity.

The basic description of supercritical fluid phase behavior which has been developing over the last few years should benefit significantly the development of commercial applications at these conditions. The use of a cosolvent can improve the economics of certain separations significantly, and extend the technology to include moderately polar species. The interest in this technology has grown rapidly based on the number of articles in the literature and patent activity, and will continue with the increasing development of specialty products and environmental applications.

Supercritical Solvent Effects on Chemical Reactions

An extremely pronounced pressure effect was discovered for the rate constant of the unimolecular decomposition of a-chlorobenzyl methyl ether in supercritical 1,1-difluoroethane. A change in pressure of only 15 bar, increased the rate constant by an order of magnitude, which corresponds to activation volumes reaching -6000 cm^3/mol. This suggests rate constants and equilibrium constants of certain reactions could be adjusted over a wide range by modest changes in temperature or pressure of the solvent in the critical region. This has important implications concerning pressure and temperature effects on oxidation reactions in supercritical water.

The characteristics of pressure effects on a reversible reaction of 2-hydroxypyridine and 2-pyridone in supercritical 1,1-diflouroethane and propane, both experimentally and theoretically, have been explored. This study complements a recent investigation of the reversible redox reaction of the I_2/I^- couple in supercritical water. Together these studies explore reactants that are nonpolar, polar, and ionic, and supercritical solvents that are nonpolar(propane), and polar(1,1-difluoroethane, and water) in order to achieve a molecular understanding of pronounced pressure effects in terms of the relevant physical and chemical properties.

A small change in the pressure near the critical point of the solvent causes a large change in density-dependent properties such as the solubility parameter, refractive index, dielectric constant, and solvatochromic polarity parameter. This suggests that pressure may be used to manipulate reactions in supercritical fluids, based on a thermodynamic solvent effect.

Supercritical fluid 1,1-difluoroethane can be used to adjust K_c for the tautomerization of 2-hydroxypyridine and 2-pyridone over a continuum from gas phase values to those encountered in polar liquid solvents, isothermally over a relatively small pressure range. It was changed by a factor of four with a small change in pressure. This adjustability is consistent with results obtained for the unimolecular decomposition of a-chlorobenzyl methyl ether[1], the $\Delta\bar{v}_{rxn}$ for the electrode reduction of I_2 to I^- ,[2] the Diels-Alder reaction of maleic anhydride with isoprene, and the parallel Diels-Alder additions of methylacrylate and cyclopentadiene.

In another study, an alumina electrochemical cell containing an ultramicroelectrode was used to make precise voltammetric measurements in supercritical water. The effect of pressure on the redox potential of the I_2/I^- couple was measured from 230 and 300 bar at 385 °C. The partial molar volume change for the reduction of I_2 to I^-, $\Delta\bar{v}_{rxn}$, is pronounced, mostly because of the interplay between strong electrostatic forces and the large isothermal compressibility of the fluid. A modified Born model together with a perturbed hard sphere equation of state predict the data accurately above 265 bar with no adjustable parameters. At lower pressures, ion pairing is thought to reduce the magnitude of $\Delta\bar{v}_{rxn}$. These large pressure effects on solvation free energies may be used to manipulate reaction equilibria and reaction mechanisms in supercritical water.

Wastewater and Sludge Experiments

In addition to the wide variety of SCF experiments conducted by Johnston and his graduate students, a series of wastewater treatability studies are currently underway. The objective of these experiments is to improve SCWO designs. Some factors that influence the designs of SCWO systems are: oxidation kinetics; heat transfer; corrosion, erosion, encrustation, charring, instrumentation, downtime, start-up, performance of materials; safety; and a host of transport and separations considerations.

SUMMARY

Supercritical water oxidation presents a potentially viable process for the destruction of toxic wastes, wastewaters, and sludges. Similarly, subcritical water oxidation processes offer opportunities for conditioning certain wastewaters and treatment plant sludges.

The deep-well subcritical and supercritical technology presents a number of interesting possibilities. The process has scientific validity and offers reasonable promise for operational success. Research and field tests are expected to address a variety of technical questions.

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ALBERTA'S APPROACH TO HAZARDOUS WASTE MANAGEMENT

by

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The Alberta Special Waste Treatment Centre, complete with high temperature incineration, physical/chemical treatment and secure landfill is located near the Town of Swan Hills and is the hub of a waste management system which includes collection, transfer and controlled transportation of hazardous wastes within Alberta.

The management system is owned by a Joint Venture of private industry and the Alberta Special Waste Management Corporation, a Crown Corporation set up to ensure safe and efficient management of hazardous wastes in the province.

The successful siting of the treatment centre is a first for North America where the usual reaction from local citizens has been "not in my backyard". By using open dialogue with the public and establishing trust and confidence, the siting process actually resulted in communities competing for the project.

The $50 million (Canadian) Treatment Centre employs 60 people and is capable of treating and disposing of any hazardous waste produced in Alberta excepting explosives and radioactive materials. Initial design capacity is 15,000 Tonnes/year with provision for expansion. Monitoring of the air, surface and groundwater, soil and vegetation is an ongoing requirement of the facility.

1. BACKGROUND

The opening of the Alberta Special Waste Treatment Centre on September 11, 1987 was a milestone without precedent in North America. Nowhere else on the continent is there a fully integrated facility which is capable of treating and disposing of all known special wastes at one location. And nowhere else in North America has a society been more involved in identifying the problem of managing hazardous wastes in working towards a solution.

2. SITING

The process which led to the construction of the treatment centre began in the late 1970s when Albertans began asking themselves questions about the nature of the problem and possible solutions. Public hearings were held in the early years of this decade. The committees which toured the province in search of answers submitted sound, well-considered recommendations to the Alberta government which reflected the concerns of many Albertans. When it was agreed that a special waste treatment facility should be built in the province and that government should facilitate the development of a special waste management system involving private sector expertise, then more public hearings were conducted to identify a suitable site.

There was debate, and at times controversy, about where, when and how such a facility should be built, and importantly, there was public participation. Community and special interest groups were consulted and municipal councils were invited to participate in the process. Information was provided to the many who asked.

Several Alberta communities responded to the call to be host of a special waste treatment centre. Each met the necessary environmental criteria and each had held a plebiscite that indicated that the majority of citizens were in favor of hosting the facility. On March 10, 1984 the cabinet of the Alberta government chose a site near Swan Hills.

3. INFRASTRUCTURE DEVELOPMENT

Soon after the selection of the Swan Hills site several other significant events took place. The Alberta Special Waste Management Corporation (ASWMC) was created by the Alberta government on April 1, 1984. This crown corporation was created to "deal adequately with hazardous waste that is not being adequately dealt with by other management facilities". One of the first tasks of ASWMC was to provide services to the site to enable construction of the plant to begin. 10 km of road to the site was rebuilt and necessary supplies of power, natural gas, telephone and industrial water were provided.

The ASWMC entered into a Joint Venture with Bow Valley Resource Services Ltd. to develop and own the facility. Chem-Security Ltd. of Calgary, Alberta was contracted to build and operate the plant using world class state-of-the-art technology.

4. PLANT DESIGN

Design of the facility required knowledge of both the wastes requiring treatment and the standards demanded by the regulatory agencies and local community.

Inventories of hazardous waste in Alberta suggested that up to 100,000 Tonnes is generated each year. Based on European experience a ratio of 1/4 was applied to arrive at the amount of waste which could be available for off-site treatment. Therefore the initial design capacity was set at 25,000 Tonnes in two phases. Phase I included 11,000 Tonnes of organic capacity and 4,000 Tonnes of inorganic capacity.

In designing the facility, four guidelines were established:

- Build Small - can always add, but cannot easily reduce capacity.

- Build Flexible - should plan facilities in such a way as to allow them to be easily modified/expanded, to accommodate additional or new waste streams.
- Build Modular - as in the auto industry, should standardize as much as possible on facility components to ensure easy access to replacement or additional parts and, of course, design and engineering costs if the plant is expanded.
- Review Options - are entering a period of numerous and increasing technological advances in the field of waste treatment and should maintain the development constantly to take advantage of new and innovative techniques and equipment - keep an open mind.

5. THE INCINERATOR

The Facility design brief calls for a 17.5 Million BTU/hour high temperature incineration capacity with the flexibility to increase that to 24 Million BTU/hour.

This posed an interesting problem for the project group because traditionally hazardous incineration has been accomplished by means of rotating kilns, sized to enable the kiln to accept 45 gallon (210 litre) drums - the most accessible, and therefore popular form of container for the collection and shipment of wastes.

It is also generally agreed that a kiln of approximately 2 metres internal diameter is required to safely incinerate drums - due to mechanical considerations - and kilns of such a size have usually a far greater capacity than required for the volume of wastes to be incinerated in the Facility.

The solution was found through an innovative variation of the rotating kiln design by the Swiss engineering company, Von Roll of Zurich.

Their answer was to develop the "rocking kiln system", a compact version of the rotating kiln that agitates the waste in the primary combustion chamber by a rocking rather than rotating motion.

In terms of operating characteristics the "rocking kiln system" is identical to its rotating cousin save for the "batch" discharge of residuals - all other systems, loading, fuel, air pollution control train, stack, etc. are essentially the same.

The key element of the system is the rocking kiln furnace designed to provide controlled incineration of wastes in containers or drums without any pre-treatment. A process computer automatically controls the combustion process and keeps the thermal output of the system constant.

The kiln turns on roller bearings. The rocking motion produces a stoking effect which brings the oxygen needed for the incineration into close contact with the waste matter on the bottom of the kiln, thus ensuring its rapid and complete incineration. The kiln swivels on its own axis at an angle of approximately 90°.

The rocking kiln action is hydraulically driven. The slag is removed at intervals by swinging the drum through 180°. Now the flue-gas opening which in normal operation is situated at the top serves as outlet of the slag which falls into the wet slag discharger. Here the slag is quenched in a water bath and carried out. Throughout the slag discharging the incineration process by the liquid waste burner continues uninterruptedly.

The flue-gases leave the kiln, as required, at a temperature of 1000°C to 1300°C and enter the secondary combustion chamber. Should the temperature of the flue-gases fall below the required 900°C-1200°C, depending on the category of waste, a standby burner automatically switches on so as to maintain the gases in the secondary combustion chamber above 1200°C.

At this point the flue-gas enters the pollution control train which consists of a spray cooling tower where the flue-gas flows downwards while being sprayed and cooled with water. The flue-gas will then pass through a venturi scrubber where the more "coarse" particulates are removed and, finally, into an ionizing wet scrubber where the gas is "polished" before being vented through the 20 metre stack.

The system allows for excellent operational flexibility since, at no time, need the total incineration plant be down for maintenance, and it allows for one unit to be dedicated to a specific waste stream and thereby programmed for greatest combustion efficiency, leaving the other unit to handle the variations in feedstock BTU levels attributed to routine variations in waste receipts.

6. PHYS/CHEM TREATMENT FACILITY

The phys/chem plant is designed to treat 4,000 Tonnes/year of inorganic waste based on a five day week and single eight hour shift.

The wastes are received in liquid, sludge and solid forms and the plant is able to handle a range of substances varying as widely as waste acids, heavy metals and cyanides, in varying strengths and mixtures.

The plant contains two chemical reactors with associated pumps, agitators and controls. One of the reactors and associated equipment is teflon lined to allow it to handle particularly acidic materials.

The other reactor, which is made of a synthetic material, is used for the treatment of alkalines, cyanides and sulphides.

Treatments capable of being performed include neutralization, precipitation, oxidation/reduction and the resultant sludges, floculants and precipitates are dewatered using a filter press, the filter cake being further processed at the stablization process plant and the wastewater is pumped to the wastewater treatment plant prior to injection down the deepwell.

7. STABILIZATION

The stabilization process provides for treatment of residues and effluents from other waste processes and for treatment of wastes not suitable for either incineration or inorganic treatment processes. Stabilization comprises solidification and/or fixation. Fixation is a process whereby chemically reactive compounds such as cement are mixed with the water and other components in sludges to form more stable solids such as concrete which both immobilizes and, in some cases, chemically changes the waste to render it non-toxic and landfillable.

The stabilization plant consists of a series of feedhoppers for the stabilization materials and a storage pit from which the waste to be stabilized is fed to the "pug mill" batch type mixer.

The plant staff continue to research the most appropriate stabilization medium and testing protocols. It is preferable that locally obtainable media such as cement kiln dust or coal fired plant fly ash be used.

Whilst not considered processes the landfill area and the injection deepwell are essential components of the Facility for the final disposition of residuals.

8. SECURE LANDFILL

The landfill area is comprised of a series of trenches or cells, whose location at the technically sound site, affords a natural protection of the environment from the materials that may be placed in it. In addition to the natural protection afforded by the geology of the site Facility, additional measures such as stablization of wastes and a comprehensive liner with a network of drainage systems to entrap and collect any leachate from the trench, are also a part of the secure landfill. Such leachate collection systems are constantly monitored although, since no liquids are placed in secure landfills and the active cells will be completely covered by a rigid roof/structure, it is highly unlikely any leachate can be generated.

9. INJECTION DEEPWELL

Deepwell injection is a widely used technique in the oil industry and petrochemical industry in Alberta. Liquids are injected into suitable geological formations which are isolated from any aquifer or portable water. This is a commonly used method of disposing of drilling fluids and saline wastes from oil exploration and production wells and is also used by industry where the hydrogeology is appropriate to dispose of aqueous, usually saline, process wastes. In the case of the Facility, this method of disposal is used for process wastewater, wastewater from the incinerator scrubber, site run-off water, and wash water from the truck decontamination system. The deepwell is not used for the disposal of any raw special or hazardous wastes.

The Facility deepwell has been drilled to a depth of 2,045 metres where the Winterburn Formation will be used as the injection zone. Test injection has indicated a zone capacity of up to 400 litres/minute at pressures in the order of 300 kilo Pascals.

Prior to injection, wastewater is treated to ensure compatibility with the zone.

10. PERMITS AND LICENCING

The facility is subject to approval and licencing under the Alberta Clean Air and Clean Water Acts. During the first 8 months of 1988 the incinerator underwent a comprehensive test burn program to establish and prove the destruction and removal efficiency capabilities as well as to determine emission levels for the stack gases. The incinerator is designed for 99.9999% removal of chlorinated organics and it had to show this on a number of principal organic hazardous constituants (POHCs).

The licences also address the other components of the facility.

11. CONCLUSION

Alberta feels justly proud of its accomplishments in siting and developing a comprehensive hazardous waste treatment facility. With the continued support and cooperation of generators, regulators and the public, it appears that the Alberta system will continue to be a model for other jurisdictions both in North America and around the world.

ENLÈVEMENT DES MÉTAUX LOURDS DES BOUES RÉSIDUAIRES PAR SOLUBILISATION BIOLOGIQUE

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RÉSUMÉ

La teneur en métaux lourds des boues résiduaires d'épuration restreindra vraisemblablement l'épandage agricole de plus de 50% de ces boues. Les procédés d'enlèvement purement chimiques de ces métaux se révèlent coûteux. Le procédé biologique consomme seulement 20% de l'acide utilisé dans les procédés chimiques. Des essais en cuvée ont permis de solubiliser le zinc, le cuivre et le cadmium à 96%, 75% et 50% en dix jours. En semi-continu, 62% du cuivre et 77% du zinc sont solubilisés avec 3 jours de temps de séjour et l'ajout de 3 g L^{-1} $FeSO_4 \cdot 7H_2O$. Dans le réacteur à ascension pneumatique en semi-continu avec 20% de recyclage et 4 g L^{-1} $FeSO_4 \cdot 7H_2O$, le zinc, le cuivre et le cadmium sont solubilisés à 91%, 89% et 81% en 0.75 jour. Une séparation solide-liquide rapide et économique se révèle être un problème à résoudre. La poursuite des études permettra de trouver des solutions à ce problème. L'étude de l'applicabilité du procédé aux boues aérobies est entreprise.

INTRODUCTION

Les boues résiduaires résultant du traitement des eaux usées contiennent environ 50 à 80% de la quantité totale de métaux lourds admis à la station d'épuration. Leur contenu en métaux est de l'ordre de 0.5 à 2.0% (base sèche), pouvant même atteindre des valeurs de 4%. Lors de l'épandage de ces boues pour fins de valorisation agricole, les métaux s'accumulent dans les sols et présentent par la suite des risques de toxicité pour les humains subséquemment à l'accumulation successive dans les plantes et les animaux (Lester et al., 1983). Le cuivre, le zinc et le nickel peuvent devenir inhibiteur de la croissance des plantes. Davis et Carlton-Smith (1984) ont démontré que le cuivre est phytotoxique à plus de 105 mg Cu kg^{-1} de sol à un pH de 7.0, le zinc l'est à 319 mg Zn kg^{-1} au même pH. Au Québec, l'absence de norme sur la teneur en cuivre à ne pas dépasser dans le sol est inquiétante, surtout q'au Québec c'est l'élément qui dépasse le plus souvent les normes. Le cadmium est un élément très toxique pour les mammifères et l'humain (Webber, 1986; Tjell, 1986). Le plomb et le chrome inquiète moins les scientifiques car ils sont très peu mobiles et peu assimilables par les plantes (Webber, 1986).

En Ontario, plus de 50% des boues dépassent les normes de teneur en métaux lourds (Wong et Henry, 1984). Aux États-Unis, 50 à 60% des boues dépassent les normes (Wozniak et Huang, 1982). Au Québec, 18 villes sur 34 dépassaient les normes lors d'analyses faites en 1988 par le Ministère de l'Environnement du Québec (St-Yves et Beaulieu, 1988). L'enlèvement des métaux lourds des boues permettrait de réduire les risques pour la santé et l'environnement tout en autorisant un accroissement du taux d'épandage au bénéfice des municipalités et des agriculteurs.

Le procédé ici préconisé pour l'enlèvement des métaux lourds présents dans les boues est basé sur un enlèvement des métaux par solubilisation biologique, une séparation des solides de la phase liquide contaminée, et une précipitation subséquente des métaux contenus dans la solution par addition d'hydroxide de calcium (Wong et Henry, 1983). Le principal micro-organisme associé au processus de solubilisation biologique est une bactérie chémoautotrophe aérobie acidophile: *Thiobacillus ferrooxidans*. La solubilisation s'effectue simultanément selon deux mécanismes, soit directement par le métabolisme de *T. ferrooxidans* dont l'action enzymatique oxyde les sulfures de métaux mettant ainsi ces métaux en solution (réaction 1), ou indirectement par le produit de son métabolisme. Dans ce second mécanisme, les bactéries oxydent premièrement le fer ferreux (II) en fer ferrique (III) (réaction 2). Le fer ferrique, agent oxydant, oxyde ensuite les autres métaux (réaction 3) et les libèrent dans la solution. Une seconde espèce parfois impliquée dans le processus, *Thiobacillus thiooxidans*, contribue à la solubilisation en oxydant les sulfures de la boue ce qui résulte en une solubilisation des sulfures métalliques par acidification (Schönborn et Hartmann, 1978).

$$MS + 2O_2 \xrightarrow[\text{T. thiooxidans}]{\text{T. ferrooxidans}} M^{2+} + SO_4^{2-} \qquad (1)$$

$$2FeSO_4 + 0.5\ O_2 + H_2SO_4 \xrightarrow{\text{T. ferrooxidans}} Fe_2(SO_4)_3 + H_2O \qquad (2)$$

$$4Fe_2(SO_4)_3 + 2MS + 4H_2O + 2O_2 \longrightarrow 2M^{2+} + 2SO_4^{2-} + 8FeSO_4 + 4H_2SO_4 \qquad (3)$$

MATÉRIEL ET MÉTHODES

Des essais de solubilisation des métaux lourds ont été effectués successivement en cuvée de 3.0 L (Tyagi *et al.*, 1988; Tyagi et Couillard, 1987), en semi-continu sans recyclage (2.5 L) et avec recyclage (3.0 L) (Mercier, 1988), ainsi qu'à l'échelle semi-pilote dans un bioréacteur à ascension pneumatique (BAP ou air-lift) d'un volume de 30 L qui a été opéré en continu à des temps de séjour de 1.5 jours et 0.75 jour. Un réacteur en continu de 5 L a aussi été opéré pour fin de comparaison au temps de séjour de 1.5 jours.

Les essais en cuvée et en BAP ont été effectués avec des boues anaérobies provenant de la station d'épuration des eaux usées de Valcartier; ceux en réacteur de 2.5 L et 3.0 L avec des boues anaérobies de la station de Ville de Deux-Montagnes. Le tableau 1 donne les teneurs en métaux des diverses boues et les normes québécoises qui restreignent l'épandage agricole des boues. Dans les essais en cuvée, un inoculum de 10% V/V fut utilisé.

Dans tous les essais sauf ceux avec les réacteurs BAP de 30 L et continu de 5.0 L, les boues d'alimentation sont acidifiées à pH 4 avec de l'acide sulfurique ce qui requiert de 0.15 à 0.18 g H_2SO_4 par g de boue sèche. Dans les essais avec le BAP et le réacteur en continu de 5.0 L l'acidification se fait jusqu'à ce qu'un pH entre 4.5 et 4.8 soit atteint, ce qui requiert 0.08 g H_2SO_4 par g de boue sèche.

Les essais en semi-continu (2.5 et 3.0 L) ont été faits avec l'addition de 1 g L^{-1} de $FeSO_4 \cdot 7H_2O$ étant donné que la boue de Ville de Deux-Montagnes n'était pas apte à faire croître les thiobacilles. Un essai à 3 g $FeSO_4 \cdot 7H_2O$ par litre de boue alimentée a aussi été fait dans ce cas.

Dans le BAP de 30 L, la quantité de $FeSO_4 \cdot 7H_2O$ a été augmentée à 4 g L^{-1} pour le temps de séjour de 0.75 jour.

Tableau 1: Teneur en métaux et en solides totaux des boues utilisées en mg kg^{-1} de boue sèche

	Boues anaérobies de Valcartier	Boues de Deux-Montagnes réacteur semi-continu sans/et avec recyclage (2.5 L et 3.0 L)	Norme Québécoise (Flynn *et al.*, 1987)	
			R	O
Cu	1 600 - 2 300	898	600	1 000
Zn	640 - 1 750	750	1 750	2 500
Cd	10 - 20	3.8	10	15
% solides totaux	1.8 - 3.0	3.0		

R = recommandée
O = obligatoire

Tous les instruments venant en contact avec les boues sont en plastique, en verre ou en bois. Les métaux sont analysés selon APHA (1985) avec un spectrophotomètre à absorption atomique à flamme de type AA575 de Varian. Les souches originales de *T. ferrooxidans* (ATCC, 19859) et *T. Thiooxidans* (ATCC 19377) ont été acclimatées aux boues (Tyagi *et al.*, 1988). Pour l'analyse des métaux, la fraction liquide contenant ceux-ci était séparée des solides par centrifugation. Les essais en cuvée ont eu lieu à 24°C, dans les autres cas la température était de 30°C. La mesure du coefficient volumétrique du transfert d'oxygène (K_La) et du taux de consommation d'oxygène a été faite selon la technique d'interruption de l'aération (Ramalho, 1983).

RÉSULTATS ET DISCUSSION

Essais en cuvée

Lors des essais en cuvée, le pH passa de 4.0 à 3.0 (culture mixte), 3.1 (culture simple) et 3.5 (témoin) après 10 jours. De meilleurs taux de solubilisation furent obtenus avec l'inoculum contenant les deux espèces bactériennes. La figure 1 donne la solubilisation du cuivre en fonction du temps alors que le tableau 2 donne les pourcentages de solubilisation des divers métaux après 10 à 12 jours de réaction.

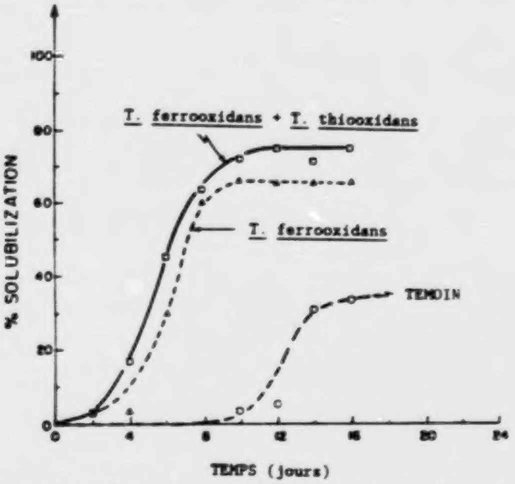


Figure 1: Solubilisation du cuivre des boues anaérobies en fonction du temps de réaction en cuvée (Tyagi *et al.*, 1988).

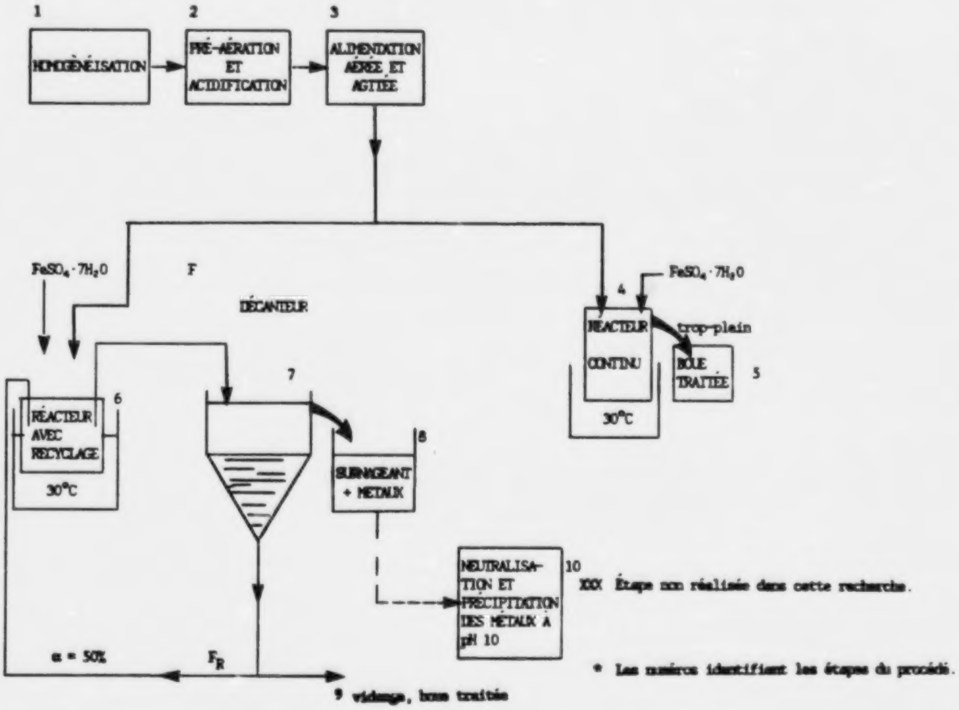
Tableau 2: Solubilisation maximale obtenue après 10 à 12 jours de réaction en cuvée (Tyagi *et al.*, 1988).

	Pourcentage de la concentration initiale des métaux		
	Zn	Cd	Cu
Témoin	80	25	35
T. ferrooxidans	90	40	65
T. ferrooxidans + *T. thiooxidans*	97	50	75

De ces résultats, il apparait que la boue contenait déjà des thiobacilles car le pH s'abaisse à 3.5 sans qu'il y ait inoculation (témoin). L'inoculum mixte donne un meilleur rendement que *T. ferrooxidans* seul. Le zinc solubilise facilement (témoin), le cuivre un peu moins et le cadmium encore moins. Tyagi *et al.* (1988) ont aussi démontré que le pourcentage de solubilisation diminuait avec l'augmentation du contenu des boues en solides totaux.

Essais dans un réacteur en semi-continu sans recyclage (2.5L) et avec recyclage (3.0 L)

La figure 2 montre le schéma global du procédé. Les deux réacteurs étaient opérés simultanément à partir de la même boue. Le tableau 3 présente les valeurs moyennes des divers paramètres lors des différents essais. La figure 3 permet de voir le pourcentage de solubilisation en fonction du temps de séjour pour le cuivre et le zinc.


1
HOMOGÉNÉISATION
2
PRÉ-AÉRATION ET ACIDIFICATION
3
ALIMENTATION AÉRÉE ET AGITÉE
$FeSO_4 \cdot 7H_2O$
F
DÉCANTEUR
4
$FeSO_4 \cdot 7H_2O$
RÉACTEUR CONTINU
30°C
trop-plein
BOUE TRAITÉE
5
6
RÉACTEUR AVEC RECYCLAGE
30°C
7
8
SURNAGEANT + MÉTAUX
10
NEUTRALISATION ET PRÉCIPITATION DES MÉTAUX À pH 10
α = 50%
F_R
9 vidange, boue traitée

XXX Étape non réalisée dans cette recherche.

* Les numéros identifient les étapes du procédé.

Figure 2: Schéma global du procédé de solubilisation des métaux dans les boues anaérobies

Tableau 3: Valeurs moyennes du taux de solubilisation, du POR, du pH et de la saturation en O_2.

Concentration de $FeSO_4 \cdot 7H_2O$ ($g\ L^{-1}$)	Temps de séjour (jours)	Taux de solubilisation ($mg\ L^{-1}\ jour^{-1}$) Zn		Cu		POR alimentation (mV)	POR (moyen) (mV)			pH moyen		% saturation O_2		Taux aération VVM[4]	
		C[1]	R[2]	C[1]	R[2]		C.[1]	R.[2]	BR[3]	C[1]	R[2]	C[1]	R[2]	C[1]	R[2]
1	1	11.3	12.2	6.2	6.9	215	442	418	301	3.01	3.09	75.8	76.0	0.7	1.1
1	2	7.4	7.1	3.7	3.5	74	330	405	305	3.22	2.98	48.0	57.9	0.5	1.0
1	3	4.9	4.8	3.5	3.1	141	436	410	289	3.10	3.12	70.4	72.8	0.7	1.0
3	3	5.3	4.3	5.2	4.2	227	523	501	331	2.84	2.91	78.1	61.0	0.5	1.0
1	4	5.4	6.3	2.5	2.3	53	325	298	275	2.97	3.26	75.6	63.1	0.7	1.0

[1] réacteur continu
[2] réacteur avec recyclage
[3] boue recyclée
[4] volume d'air par volume de réacteur par minute

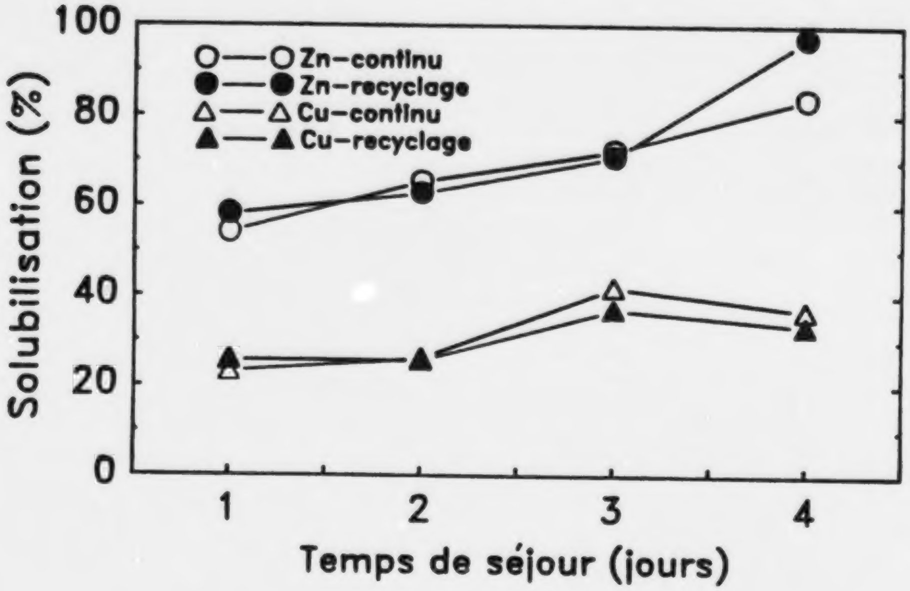


Figure 3: Effet du temps de séjour sur la solubilisation du cuivre et du zinc dans les deux types de réacteurs avec 1 g $FeSO_4 \cdot 7H_2O$ L^{-1}.

De ces courbes, il est évident que la solubilisation du zinc augmente avec le temps de séjour alors que pour le cuivre ce n'est vrai que jusqu'à 3 jours de temps de résidence. En se rapportant au tableau 3, il est plus facile de comprendre le comportement du cuivre au temps de séjour de 4 jours. En effet, à ce temps de séjour, le pH est relativement élevé dans le réacteur avec recyclage et le potentiel rédox (POR) est beaucoup plus faible que dans les autres cas; un POR minimum de 200 mV est nécessaire pour la solubilisation du cuivre alors que pour le zinc -100 mV est suffisant (Theis et Hayes, 1980). Le POR plus faible à 4 jours de temps de rétention, pourrait s'expliquer par deux facteurs. Le premier, est le POR faible dans le bassin d'alimentation (53 mV), qui semble diminuer le POR des réacteurs. Le second facteur serait l'épuisement du 1 g L^{-1} de $FeSO_4 \cdot 7H_2O$, avec 4 jours comme temps de séjour. Enfin, autre conclusion importante, le réacteur avec recyclage n'est pas plus efficace que le simple réacteur en continu.

Une expérience avec 3 g L^{-1} de $FeSO_4 \cdot 7H_2O$ a été effectuée à 3 jours de temps de séjour afin de comprendre la raison de l'efficacité moins élevé du réacteur avec recyclage. La figure 4 présente l'effet de l'augmentation de la concentration du sulfate ferreux.

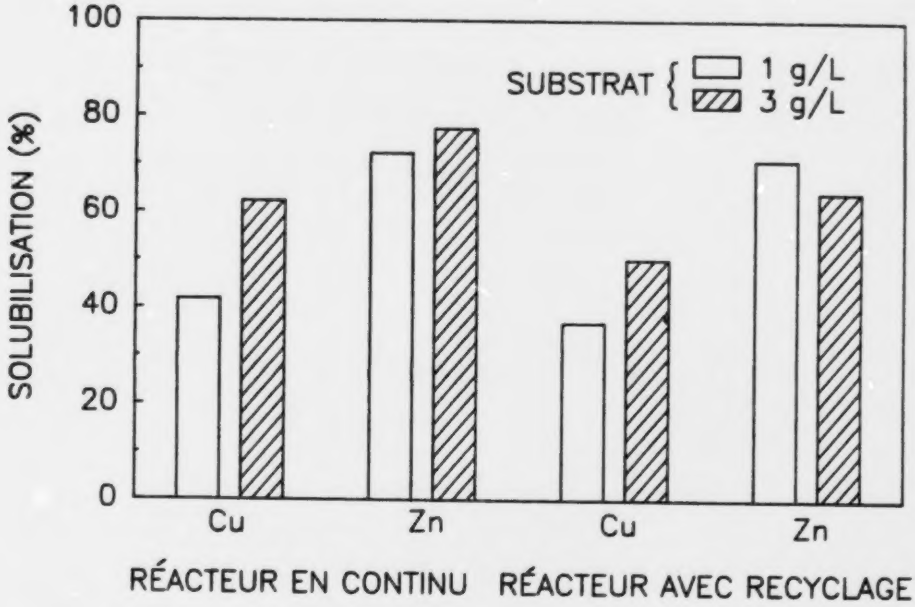


Figure 4: Influence de la concentration de $FeSO_4 \cdot 7H_2O$ sur la solubilisation du cuivre et du zinc dans les réacteurs avec un temps de séjour hydraulique de 3 jours.

Le cuivre est grandement favorisé par cette augmentation alors que l'effet sur le zinc semble incertain. Une fois de plus, le réacteur avec recyclage est moins performant. Cette différence ne peut s'expliquer par les coefficients volumétriques du transfert d'oxygène (K_La) qui sont très semblables (Tableau 4). Sur le tableau 4 et la figure 5 il apparait évident que le POR et la solubilisation du Cu sont plus faibles dans la boue recyclée alors que le pH y est plus élevé. En réalité, il y a 50.1% du cuivre solubilisé dans le réacteur alors que ce pourcentage chute à 39.8% dans le recyclage (Tableau 4). En fait, la boue étant difficile à décanter, le temps de séjour dans le décanteur a été fixé à 90% de celui dans le réacteur.

Ce long temps de décantation permet, de plus, aux bactéries d'épuiser l'oxygène dans le décanteur ce qui a pour effet d'abaisser le POR et d'augmenter le pH. La solubilisation du cuivre, plus sensible au POR est diminuée alors que la solubilisation du zinc demeure inchangée. La boue recyclée amène donc des métaux à resolubiliser dans le réacteur, ce qui a pour effet de diminuer le rendement global du réacteur avec recyclage.

Le problème actuel consiste donc à optimiser la séparation solide-liquide en vue d'éviter ce genre de problème. En résumé, il y a eu enlèvement de 62% du Cu et 77% du zinc dans le réacteur en continu sans recyclage avec 3 g L^{-1} $FeSO_4 \cdot 7H_2O$ et 3 jours de temps de rétention.

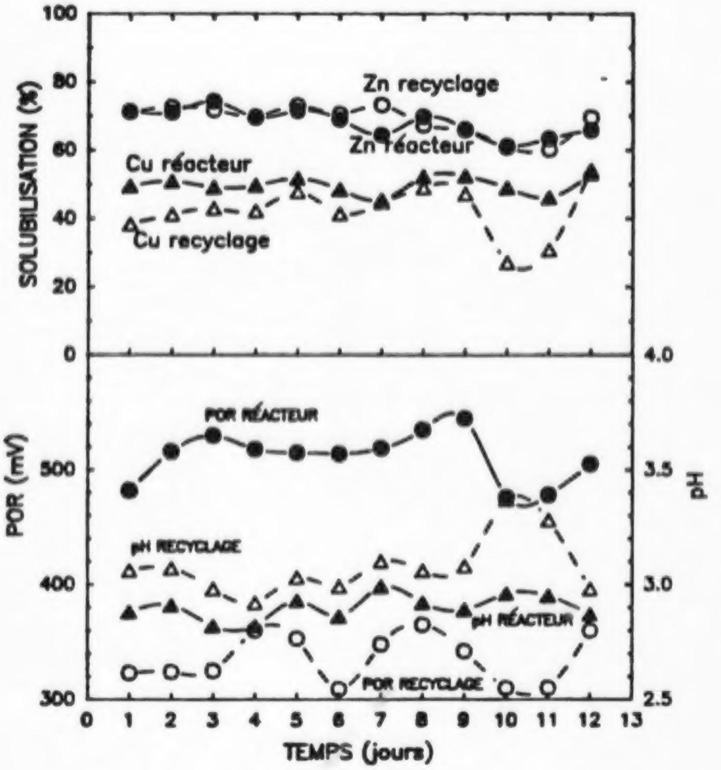


Figure 5: Pourcentage de solubilisation du cuivre et du zinc, POR et pH dans le réacteur avec recyclage et la boue recyclée au temps de séjour de 3 jours avec 3 g $FeSO_4 \cdot 7H_2O$ par litre de boue.

Tableau 4: Résultats pour les deux réacteurs à un temps de séjour de 3 jours avec 3 g L^{-1} $FeSO_4 \cdot 7H_2O$

Paramètre	Réacteur en continu sans recyclage	Réacteur en continu avec recyclage
Cu Solubilisation moyenne	62.2%	Réacteur = 50.1% Recyclage = 39.8%
Zn Solubilisation moyenne	77.4%	Réacteur = 64.1% Recyclage = 64.1%
POR (mV)	523	Réacteur = 501 Recyclage = 331
Saturation en oxygène (%)	75.6	63.1
K_La (h^{-1})	7.9	7.6
TCO (mg O_2 L^{-1} min^{-1})	0.32	0.20
pH	2.8	Réacteur = 2.9 Recyclage = 3.2

Essai avec le bioréacteur à ascension pneumatique (BAP):

La grande importance du potentiel rédox pour la solubilisation du cuivre a mené à la construction d'un bioréacteur à ascension pneumatique qui est reconnu pour sa très grande efficacité de transfert de l'oxygène. La figure 6 présente le BAP.

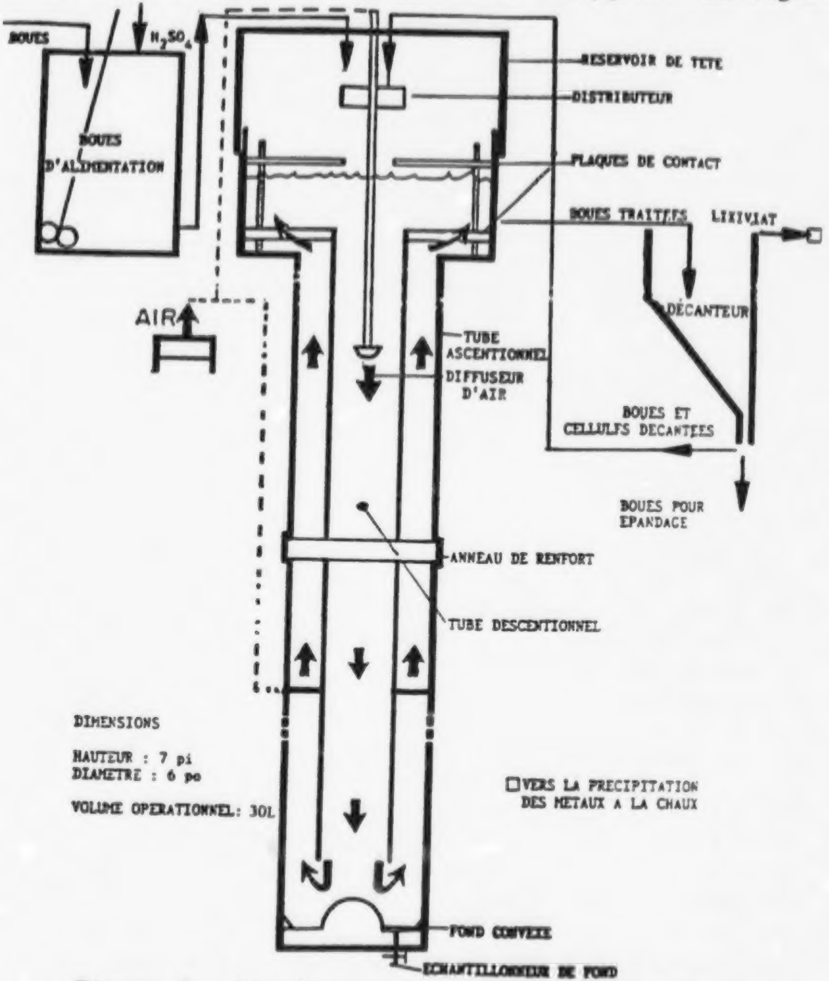

BOUES
H_2SO_4
BOUES D'ALIMENTATION
AIR
RESERVOIR DE TETE
DISTRIBUTEUR
PLAQUES DE CONTACT
BOUES TRAITEES
LIXIVIAT
DÉCANTEUR
TUBE ASCENTIONNEL
DIFFUSEUR D'AIR
BOUES ET CELLULES DECANTEES
BOUES POUR EPANDAGE
ANNEAU DE RENFORT
TUBE DESCENTIONNEL
DIMENSIONS
HAUTEUR : 7 pi
DIAMETRE : 6 po
VOLUME OPERATIONNEL: 30L
☐ VERS LA PRECIPITATION DES METAUX A LA CHAUX
FOND CONVEXE
ECHANTILLONNEUR DE FOND

Figure 6: Bioréacteur à ascension pneumatique.

Le tableau 5 présente les résultats obtenus avec le BAP de 30 L et le réacteur en continu agité de 5.0 L. La solubilisation dans le BAP à 0.75 jour est très élevée et permet d'espérer d'abaisser le temps de séjour à 0.50 jour. Par contre, les résultats dans le réacteur en continu agité ne permettent pas de déceler une meilleure performance du BAP. Les résultats de l'essai à 0.75 jour et 0.50 jour actuellement en cours permettront sans doute d'identifier le meilleur réacteur.

Tableau 5: Pourcentage de solubilisation dans les réacteurs

TEMPS DE RÉSIDENCE	RÉACTEUR EN CONTINU agité (5.0 L)	BIORÉACTEUR À ASCENCION PNEUMATIQUE (30 L)
1.5 jours (10% recyclage) (3 g/L $FeSO_4 \cdot 7H_2O$)	Cu = 77 Zn = 100 Cd 56	81 91 90
0.75 jour (20% recyclage) (4 g/L $FeSO_4 \cdot 7H_2O$)	Cu = N.D.* Zn = N.D. Cd = N.D.	89 91 82
0.50 jour		N.D.

* non-disponible actuellement

CONCLUSIONS

L'enlèvement des métaux lourds par solubilisation biologique permet d'importantes économies d'acide comparativement au procédé chimique. Le temps requis pour solubiliser le cuivre d'une façon substantielle (> 60%) est passé de 10 jours en cuvée à 3 jours en continu, à 0.75 jour dans le BAP avec un faible recyclage des boues avec 4 g $FeSO_4 \cdot 7H_2O$ par litre de boue traitée. Pour le cuivre, le potentiel rédox s'avère être un paramètre très important. La séparation solide-liquide est le principal obstacle à l'heure actuelle. Elle devra être rapide pour maintenir le POR élevé, très efficace pour concentrer les boues, et le séparateur devra résister à la corrosion. Les facteurs économiques de cette séparation seront déterminants pour l'applicabilité de ce procédé. Des analyses sur les diverses formes d'azote et de phosphore permettront de connaitre la valeur fertilisante des boues traitées. Jusqu'à maintenant, tous les essais ont été faits sur des boues anaérobies car elles contiennent des sulfures de métaux. Au Québec, la majorité des boues étant digérées de façon aérobie, la prochaine étape consistera donc à explorer l'applicabilité de ce procédé aux boues aérobies.

REMERCIEMENTS

Les auteurs adressent leurs remerciements au Centre québécois de valorisation de la biomasse (CQVB) et aux fonds FCAR (AS-2462) pour leur participation financière.

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A COMPLETE WASTEWATER TREATMENT FOR WOOD TREATING PLANTS

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Introduction

Frequently, wood treating plants employ creosote and pentachlorophenol as raw materials, which are forced into the pore structure of the wood in a pressurized cylinder. The resulting process effluent is a mixture of water, oil and toxic chemicals. Before this wastewater can be discharged, several treatment steps are required. The oil mixture must be separated for return to the process. The remaining mixture consists of water, oil and grease, phenol, pentachlorophenol (PCP), with traces of dioxin and polycyclic aromatic hydrocarbons. This wastewater must meet increasingly stringent regulatory discharge standards. Present treatment methods in many plants rely on flocculation for removal of free and emulsified oils, followed by carbon beds for removal of the toxic chemicals. The result is dischargeable water accompanied by a secondary disposal problem, i.e. carbon regeneration and/or replacement. In addition to being very costly, this method is becoming nonviable in that material containing PCP is increasingly unacceptable to waste regenerators or for landfill.

To solve this problem, Solarchem has applied its new remediation technology based on proprietary enhanced oxidation processes. The problem statement is depicted in Figure 1. Typical process water, after removal from the settling tank, still retains oil, much of it in emulsified form. During the course of our studies, we have tested water from two different wood treating plants in Eastern Canada. The water obtained from the wood treating plants often contains the following ranges of contaminants:

Oil	400 - 4000 ppm
Phenol	50 - 1000 ppm
Pentachlorophenol	20 - 300 ppm
Dioxin	15 - 300 ppb

Solarchem's enhanced oxidation processes used for this work are based on second generation photooxidation technology and related techniques which produce oxidation reactions that are several orders of magnitude faster than those using standard oxidation with O_2, O_3 and H_2O_2. The process described herein decreases contaminants by four orders of magnitude to attain required discharge levels of less than 2 ppm oils, less than 15 ppb phenol and less than 10 ppb pentachlorophenol.

In this paper, various oxidation mechanisms are reviewed. The test facilities are described, followed by a description of the test results. The Solarchem system is described and discussed. Cost factors, both direct and indirect are also discussed.

The Enhanced Oxidation Process

In enhanced oxidation processes, the principal oxidizing agents are radicals. These oxidizing agents have much greater oxidizing power than their precursors, which now perform the role of oxidant source rather than oxidizing agent. Typical oxidant sources are ozone and hydrogen peroxide. On occasion, less used oxidant sources are appropriate, e.g. hypochlorite or ferrous sulphate. The synergistic effect of the UV and O_3 or H_2O_2 has been known for many years, with one of the first interpretations being given by Prengle and co-workers(1) in 1976. The understanding of this synergistic effect is now fairly widely understood and agreed upon, in broad terms, by investigators studying photooxidation mechanisms. A recent discussion of the topic is given in papers by Glaze(2) and Peyton(3).

For some compounds, direct photolysis is a significant means of decomposition. Even where the UV is absorbed directly by the contaminant, it may be more cost-effective to treat the wastewater by radical initiated oxidation plus photolysis. In addition, UV alone may not be effective in the decomposition of intermediates which result from the initial decomposition.

Table 1 gives some reactions by which waterborne contaminants can be oxidized. Thermodynamics tells us that virtually any organic contaminant, in the presence of ozone, hydrogen peroxide or oxygen will, given enough time, be fully mineralized to CO_2, H_2O (and Cl in the case of a chlorinated compound). However, the oxidation rates for such processes are typically too slow to be of broad utility for wastewater treatment.

However, if the contaminated water stream with the O_3 and/or H_2O_2 and/or O_2 is also exposed to short wavelength ultraviolet light ($\lambda < 300$ nm), then several additional decomposition reactions are initiated which greatly increase the rate of oxidation of the contaminant.

Many contaminants which adsorb UV light dissociate into fragments called radicals, which typically react very rapidly with oxygen to form peroxy radicals (e.g. $HCOO_2\cdot$) which themselves can participate, in a complex way, with the oxidation of the contaminants present or, alternately, decompose into simpler molecular fragments.

Some contaminants do not dissociate to a significant extent upon adsorption of UV light, but instead form a relatively long lived reactive intermediate called a "triplet", as shown in Table 1. The triplet species thus formed have sufficient lifetime to encounter and react with any oxidizing species (O_2, H_2O_2, O_3, $HO\cdot$) that are present in solution, yielding as products more oxygenated compounds, which themselves will be subject to all the oxidizing processes underway in the water stream in question. If sufficient time is allowed, the contaminants will be fully mineralized.

TABLE 1

OXIDATION REACTIONS

Direct Oxidation

$$R \xrightarrow{O_3/H_2O_2/O_2} \left[\begin{array}{l}\text{Oxygenated}\\ \text{Intermediates}\end{array}\right] \xrightarrow{O_3/H_2O_2/O_2} CO_2 + H_2O$$

Photooxidation

$$R_1R_2 \longrightarrow {}^1R_1R_2^* \xrightarrow{\text{dissociation}} R_1\cdot + R_2\cdot$$

Where R_1 & R_2 are radical species such as: H, Cl, alkyl, aryl, acetyl, etc.

$$R\cdot + O_2 \longrightarrow RO_2\cdot \xrightarrow{\text{dissociation}} \left[\begin{array}{c}\text{Smaller molecular}\\ \text{fragments}\end{array}\right] \xrightarrow{h\nu/O_3/H_2O_2/O_2} CO_2 + H_2O$$

$${}^1R_1R_2^* \longrightarrow {}^3R_1R_2^* \xrightarrow{\text{dissociation}} R_1\cdot + R_2\cdot$$

$${}^3R_1R_2^* \xrightarrow{O_3/H_2O_2/O_2} \left[\begin{array}{l}\text{Oxygenated}\\ \text{Intermediates}\end{array}\right] \xrightarrow{h\nu/O_3/H_2O_2/O_2} CO_2 + H_2O$$

Photolysis of Ozone

$$O_3 \xrightarrow[H_2O]{h\nu} 2HO\cdot + O_2$$

Photolysis of Hydrogen Peroxide

$$H_2O_2 \xrightarrow{h\nu} 2HO\cdot$$

Radical Attack

$$R + HO\cdot \longrightarrow R\cdot + H_2O$$

In addition to direct photooxidation of the contaminants, the use of UV light has a powerful additional benefit, which is the formation of extremely strong oxidizing agents such as the hydroxyl radical (HO·). Once formed the hydroxyl radical will result in the rapid oxidation of the contaminants present. The potential importance of the HO· radical can be realized by consideration, for example, of the fact that HO reacts with phenol $\sim 10^8$ (one million) times as fast as does molecular ozone. These processes are illustrated in Table 1.

In all the mechanisms described, optimization of the dosage is essential as there is competition for the UV light between the contaminant and the O_3/H_2O_2 and, as well, O_3 and H_2O_2 can be traps for the HO· radical.

Using enhanced oxidation, a wide range of organic and inorganic compounds can quickly be oxidized, leading to the possibility of water decontamination by complete mineralization of the contaminant(s) to carbon dioxide and water, or in the case of chlorinated compounds also to the chloride ion in water, which is readily disposable. Intermediates in these processes are generally less stable than the original compound(s) and are thus easily oxidizable. The mechanisms outlined above are particularly attractive in environmental remediation technology since it produces no secondary disposal problem.

Complete detailed breakdown pathways for specific chemicals are generally not elucidated, even though the process is understood overall. Chlorinated compounds, such as pentachlorophenol and dioxins, are particularly well suited to these enhanced oxidation processes. As an example, Table 2 has a detailed breakdown of the steps in the mineralization of perchlorethylene, as proposed by Proksch et al (4).

Table 2.

REACTION MECHANISM FOR HYDROXYL RADICAL INIATIATED OXIDATION OF PERCHLOROETHYLENE

$$CCl_2 - CCl_2 + HO^{\bullet} \longrightarrow HOCCl_2 - {}^{\bullet}CCl_2$$

$$HOCCl_2 - {}^{\bullet}CCl_2 \longrightarrow COCL - {}^{\bullet}CCL_2 + HCl$$

$$COCl - {}^{\bullet}CCl_2 + O_2 \longrightarrow COCl - CCl_2OO^{\bullet}$$

$$2COCl - CCl_2OO^{\bullet} \longrightarrow O_2 + 2COCl - CCl_2O^{\bullet}$$

$$COCl - CCl_2O^{\bullet} \longrightarrow {}^{\bullet}COCl + CCl_2O$$

$$CCl_2O + H_2O \longrightarrow CO_2 + HCl$$

$${}^{\bullet}COCl + H_2O \longrightarrow {}^{\bullet}COO^- + H^+ + HCl$$

$${}^{\bullet}COO^- + O_2 \longrightarrow {}^{\bullet}O_2^- + CO_2$$

Test Facilities and Experimental Program

The System Process Flow Diagram of Figure 2 generally represents the photooxidation and enhanced oxidation processes which have been used in this work. The Solarchem test laboratory utilizes several pilot scale test units, one of which is comprised of three identical reactors, each of which contains a Solarchem lamp which emits strongly at wavelengths less than 300 nm. All of the work reported here has been carried out in a batch mode operation so that the results can be used for scale-up purposes. Batch sizes ranged from 70 liters to 200 liters.

Referring again to Figure 2, one or more reactors are required for a test run. Each reactor requires one or more oxidant source inputs. As described in the previous section, these oxidant sources are the precursors to formation of the actual agent of oxidation, which are fast acting radicals such as the hydroxyl radical. Liquid oxidant sources are fed to the individual reactor subsystem under metering pump control. Solarchem has also used a proprietary catalyst, application of which will be described in the next section.

The rate of supply of oxidant sources is a critical factor in rate of destruction of the compound and its byproducts. Rate of scavenging of the oxidizing radicals by the organic compounds present depends on the particular compound and also on its concentration.

The degree of effectiveness of the individual reactor subsystem can be influenced positively or negatively by engineering design and also by supply of materials for the reaction. We have found that due to the complexity of the interactions of various factors influencing the outcome of a water treatment program, theoretical considerations alone are insufficient as a basis for design. Therefore with each new water stream, a test program must consist of several runs under different conditions. The test program is an iterative method of optimizing the treatment suited to the particular situation.

The analytical instrumentation used was GC/FID, HPLC with variable UV detector, and GC/ECD for phenol, pentachlorophenol and dioxin respectively. Phenol was analyzed by direct injection down to approximately 10 ppm. At phenol levels below 10 ppm, an acetylation and extraction step was used prior to analysis. PCP was analyzed by direct injection and dioxin was extracted with toluene prior to injection.

Treatment System Overview

This testing program has resulted in presentation of a specific system design to a company operating a wood treatment plant in Ontario. Installation of the equipment is expected to occur in early 1989.

In Figure 3, both the new and the existing treatment plant are shown. One feature of the new system is that oil/water separation is an essential pre-treatment process. The oil content of this water is both heavy and light oils, much of which is finely emulsified. The first step occurs in a pair of settling tanks, used alternately and each holding about 5 days of process water. Although considerable excursions are normally expected, the oil content of water drawn after a correctly operated settling process is well below 1000 ppm. The settled oil is recycled to the process. Without further separation it would be expected that the oils would consume a large part of the dosage of the oxidants, thus making the process unnecessarily expensive. This has been confirmed by our experimental testing when the test batch was taken directly from a gravity settling process without further oil removal. Therefore, the oil/water separation shown in the Figure 3, is a two step process. The second step uses a Lamella-type gravity separator, with polymer flocculants, which produces an output oil level of 20-50 ppm of oil. The final pre-treatment is the addition of ENOX 230, a proprietary catalyst. At that point, the water is ready for presentation to the enhanced oxidation subsystem. The new system would require less than $25 per 1000 imperial gallons compared to the existing system which requires $140/1000 imperial gallons to operate. The general flow schematic for the existing water treatment system is also shown in Figure 3.

Enhanced Oxidation System Test Results and Discussion

The performance specifications for the Solarchem system are given in Table 3. As can be seen from this table, the new system must be able to achieve greater than 99.99% removal of the phenol and pentachlorophenol.

TABLE 3

SPECIFICATION FOR TREATMENT OF EFFLUENT FROM A WOOD TREATING PLANT

	Pre-Treatment	Post-Treatment
Phenol	250 ppm	<15 ppb
Pentachlorophenol	200 ppm	<10 ppb
Oil & Grease	4000 ppm	< 2 ppm

In Figure 4, a typical sample starting level of phenol was 300 ppm, and the end point was measured at 7 ppb. The PCP start point was 190 ppm and end point was 10 ppb. In this figure the normalized concentration is plotted against cumulative ozone dose. In the series of figures presented in this section, cumulative ozone dose is used as a general indicator of supplied treatment materials. Other oxidants are supplied simultaneously and their rate of consumption follows the ozone dose. The result of Figure 4 shows that PCP and phenol are removed concurrently, with PCP lagging slightly in removal rate.

Figures 5(a) and 5(b) display the same data as Figure 4 for phenol only, on linear and log scales. When the y axis is plotted as log (C/Co), it is easy to see quickly that the contaminant concentration has been reduced by over 5 orders of magnitude. There is evidence that there are two stages of removal, each with its own rate. While the oxidation processes have complete mineralization to carbon dioxide and water, for example, as the end point, the process must proceed through a series of byproducts. Generally, these intermediates are less stable than the original contaminant so these degrade more quickly. One interpretation of the shape of the curve of Figure 5(b) is that when the byproducts have mostly been oxidized, the remainder of the removal of the phenol proceeds more rapidly. In Figures 5(b) and 5(c), the initial phenol concentrations are 300 ppm and 65 ppm respectively. Both runs result in a similar shape of log plot, suggesting that the removal mechanism is similar for both lower and higher concentrations, when both are the same order of magnitude. However, there is clearly less byproduct competition for the oxidizing radicals when the initial concentration of the phenol is lower. In Figure 6, the low concentration phenol test run starts at 70 ppm and the high concentration phenol run starts at 1000 ppm. Phenol reduction is clearly more rapid when byproduct competition is not a major trap for the oxidizing radicals.

Figure 7 shows the effect of pre-treatment with ENOX 230 before oxidation. This pre-treatment greatly improves the removal of PCP, especially in the lower concentration range where the reduction is proceeding toward 10 ppb. Without ENOX 230, it is doubtful that the last order of magnitude of removal could be attained, since the curve is becoming flattened as the concentration decreases. The ENOX 230 pre-treatment neither enhances nor degrades the removal of phenol.

Figures 8(a) and 8(b) show the effect of retained oil (200 ppm) on the removal of phenol and PCP, compared to reduced oil (40 ppm). Phenol removal is slowed more and more as the concentration decreases, with apparent flattening in the region around 100 ppb. Initial concentration of phenol is at the specification level of 250 ppm. PCP removal is also slowed by retained oil but it appears that the final specified concentration of 10 ppb is still attainable for increased dosage of oxidant sources of about 50%.

Whenever pentachlorophenol is present in a process, there will also be the dioxin impurity in the PCP. While there is no specification for dioxin removal in the present work, the removal of the potentially most hazardous waste by the Solarchem enhanced oxidation system is also a requirement. In this work, we have monitored the octachloro-dibenzo-p-dioxin (OCDD) as a surrogate for all the other dioxins. The nature of these oxidation processes is highly favourable for the removal of all dioxins. In Figure 9 the OCDD removal is shown to proceed from a starting level of 240 ppb to below 3 ppb on a logarithmic scale. Further destruction of the dioxin could not be accurately measured, as the detection limit of our present analytical method is about 1 ppb.

As outlined in earlier sections, the correct choice of oxidant dose and reactor design is critical. The effect of optimization of oxidant dose is seen in Figure 10. In the case of the run labelled First Test, the specified post-treatment concentration would be attained only after supplying many times the dose of oxidants actually needed. In Figure 11, the phenol removal rate is seen to be greatly influenced by the type of reactor employed.

Conclusions

Enhanced oxidation technology has resulted in the development of a solution for a particularly difficult process water effluent problem. With this technology the post-treatment levels of 15 ppb, 10 ppb and 2 ppm of phenol, pentachlorophenol and oil respectively can be met. The operating cost is less than $25/1000 imperial gallons. This is to be compared to present operating costs of $140/1000 imperial gallons. The present costs have the potential for escalating greatly since landfill disposal of PCP contaminated materials is becoming increasingly difficult and regeneration of carbon is in the same position.

The wood treating plant effluent treatment system developed by Solarchem is not difficult to operate, and is inherently capable of 24 hour per day unattended operation. With properly chosen components and materials, the system has a high maintainability and availability.

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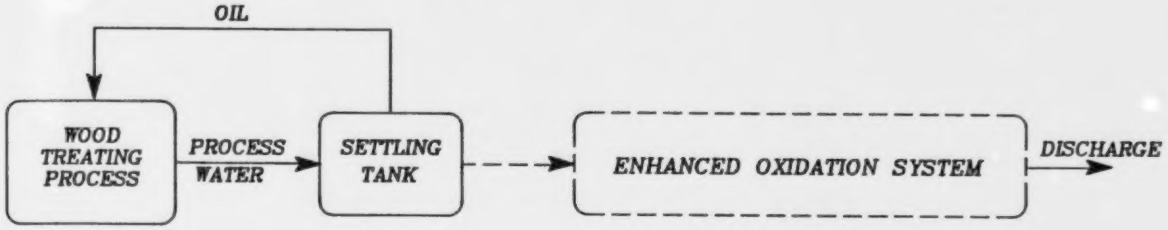
4. Proksch, E., Gehringer, P., Szinovatz, W. and Eschweilerlt, Radiation Induced Decomposition of Small Amounts of Perchlorethylene in Water, Appl. Radiat. Inst., Volume 38, No. 11, p. 911 (1987).

DEFINITION OF THE WOOD TREATING PROBLEM


OIL
WOOD TREATING PROCESS
PROCESS WATER
SETTLING TANK
ENHANCED OXIDATION SYSTEM
DISCHARGE

OIL	1000 – 4000 ppm	REDUCTION > 4 ORDERS OF MAGNITUDE	< 2 ppm
PHENOL	100 – 1000 ppm		< 15 ppb
PENTACHLOROPHENOL	50 – 300 ppm		< 10 ppb
DIOXIN	20 – 100 ppb		< 1 ppb

FIGURE 1

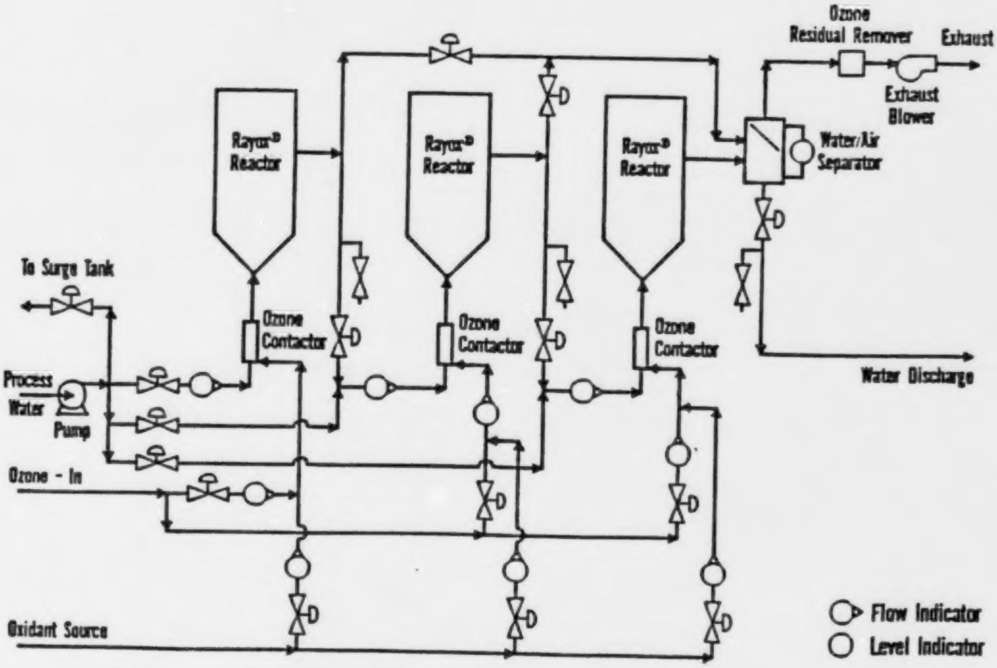
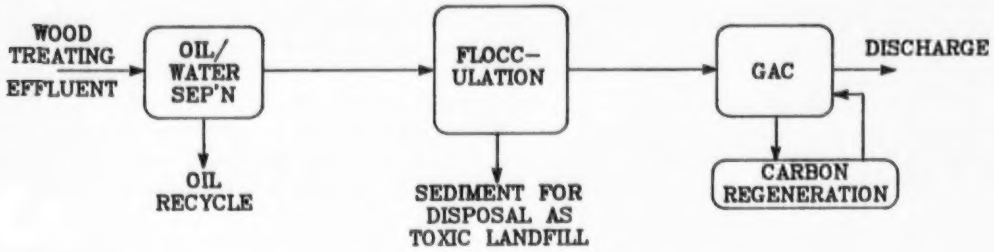

Ozone Residual Remover
Exhaust
Exhaust Blower
Rayox® Reactor
Rayox® Reactor
Rayox® Reactor
Water/Air Separator
D
To Surge Tank
Ozone Contactor
Ozone Contactor
Ozone Contactor
Process Water Pump
Water Discharge
Ozone - In
Oxidant Source
Flow Indicator
Level Indicator

Figure 2. SOLARCHEM RAYOX® SYSTEM PROCESS FLOW DESIGN

WOOD TREATING PLANT WASTEWATER SYSTEM

EXISTING SYSTEM AT AN ONTARIO WOOD TREATING PLANT


WOOD TREATING EFFLUENT
OIL/ WATER SEP'N
OIL RECYCLE
FLOCC- ULATION
SEDIMENT FOR DISPOSAL AS TOXIC LANDFILL
GAC
DISCHARGE
CARBON REGENERATION

NEW SOLARCHEM SYSTEM

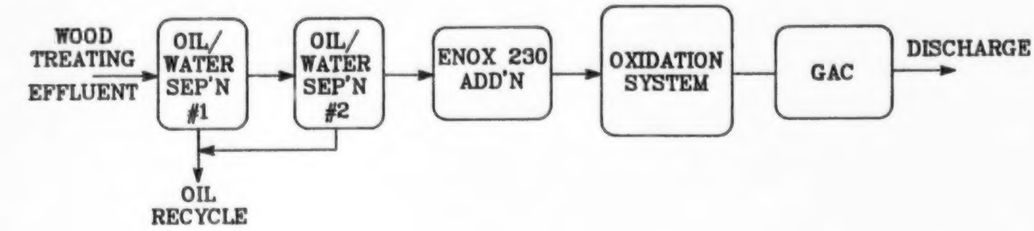

WOOD TREATING EFFLUENT
OIL/ WATER SEP'N #1
OIL/ WATER SEP'N #2
OIL RECYCLE
ENOX 230 ADD'N
OXIDATION SYSTEM
GAC
DISCHARGE

FIGURE 3

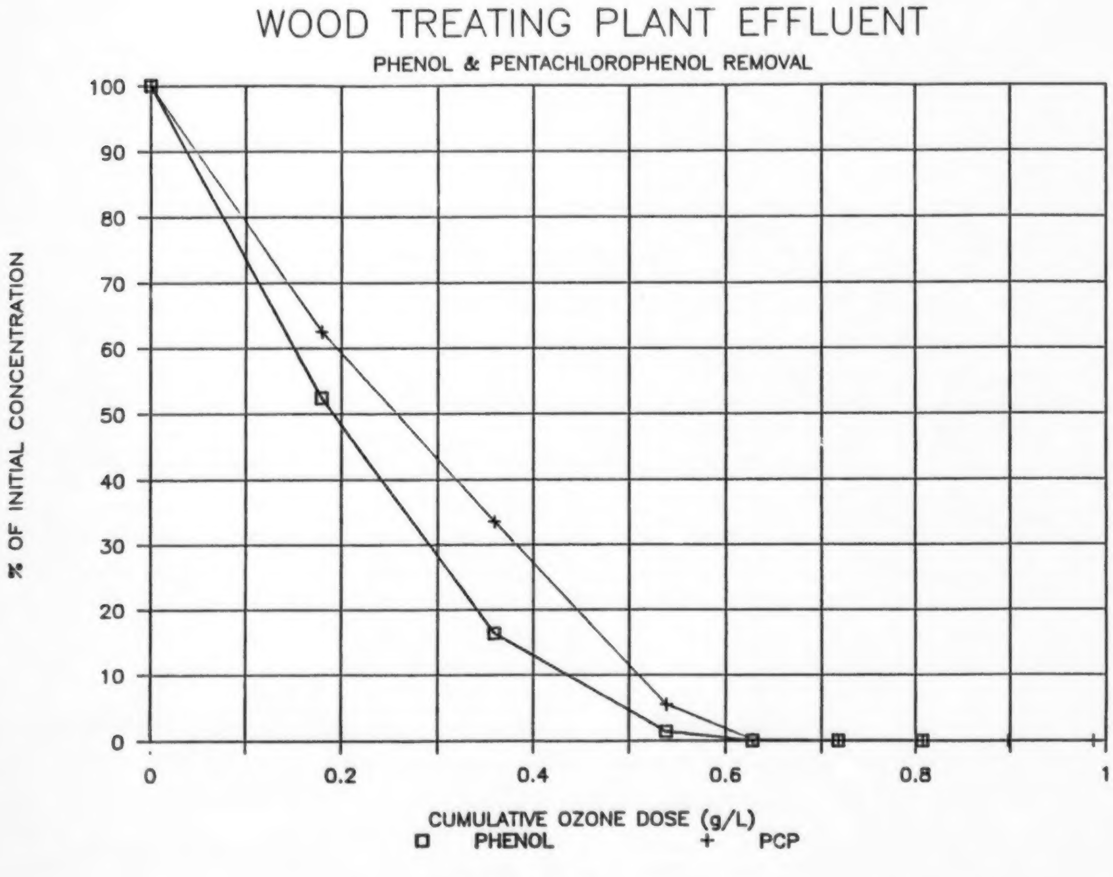


FIGURE 4

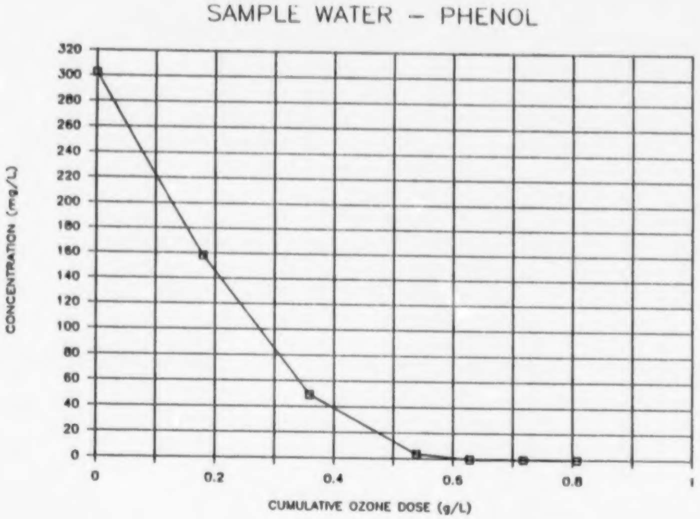


FIGURE 5(a)

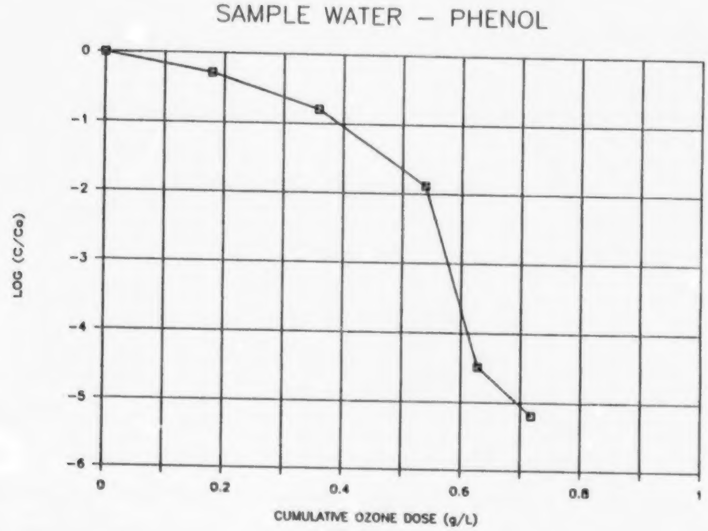


FIGURE 5(b)

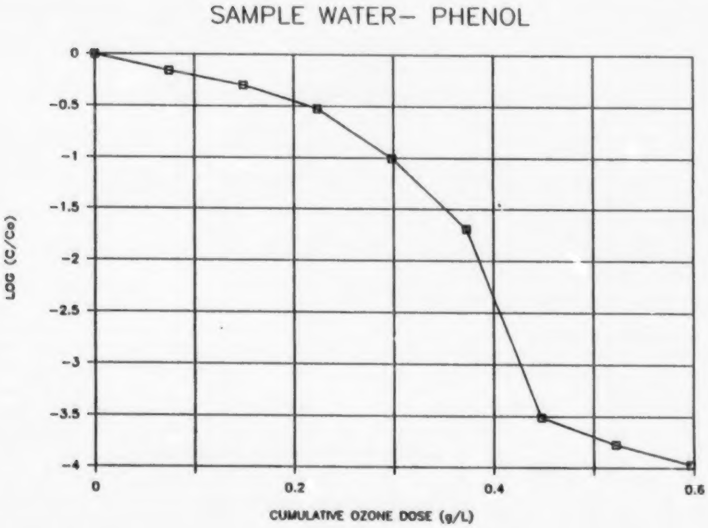


FIGURE 5(c)

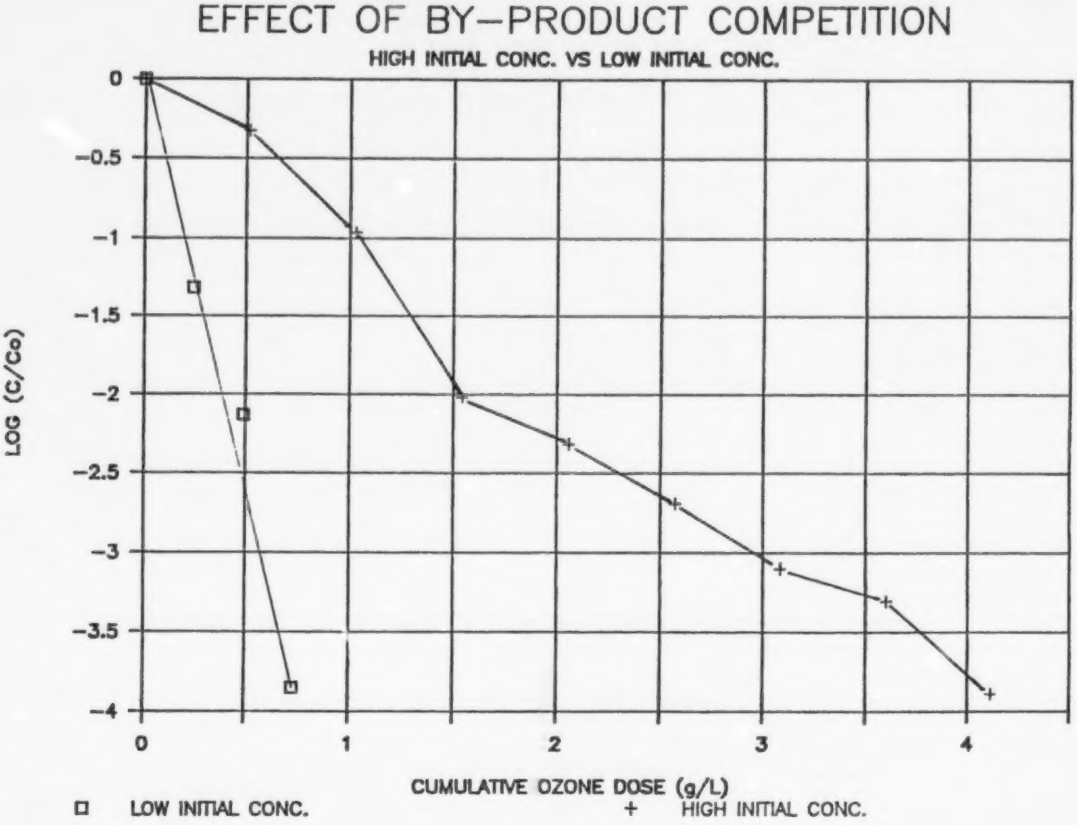


FIGURE 6

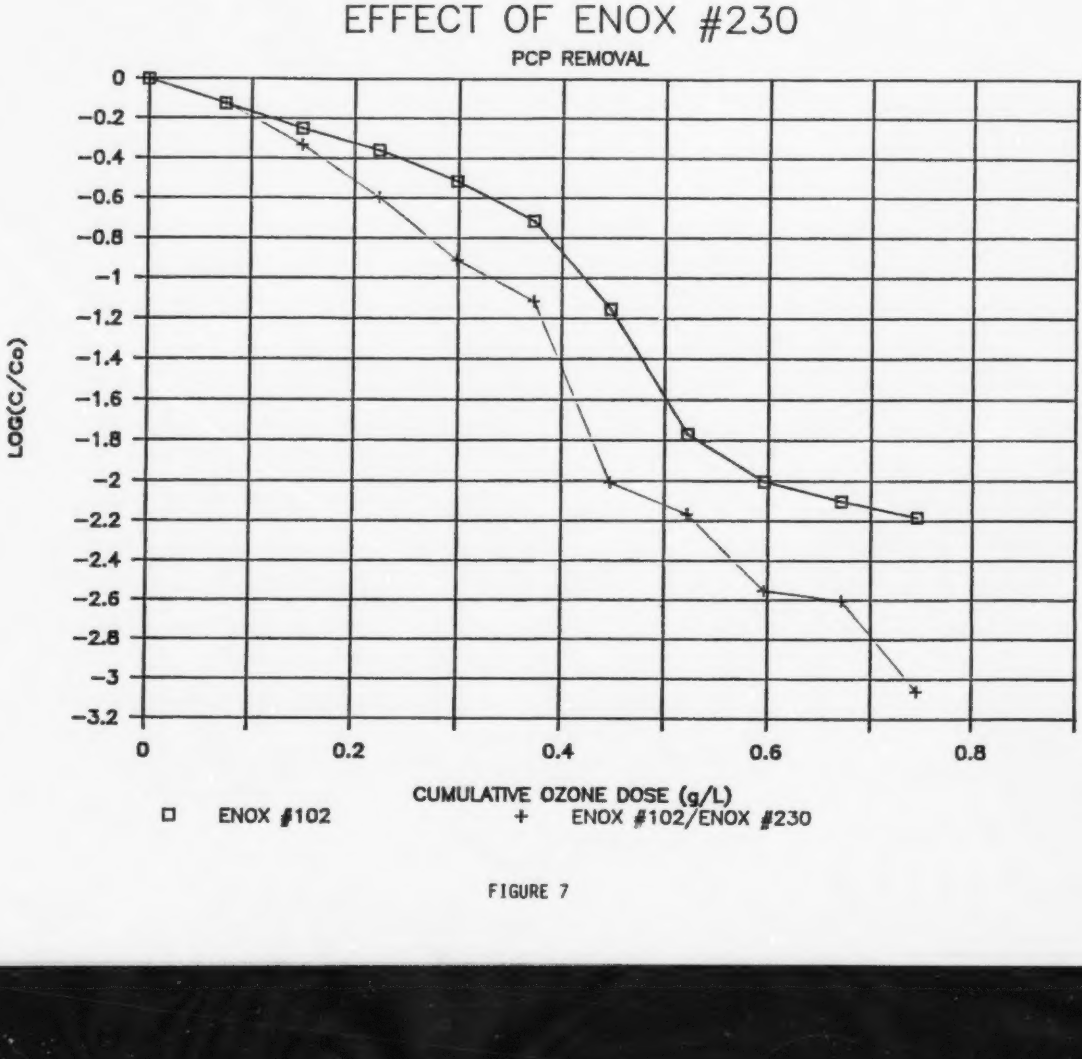
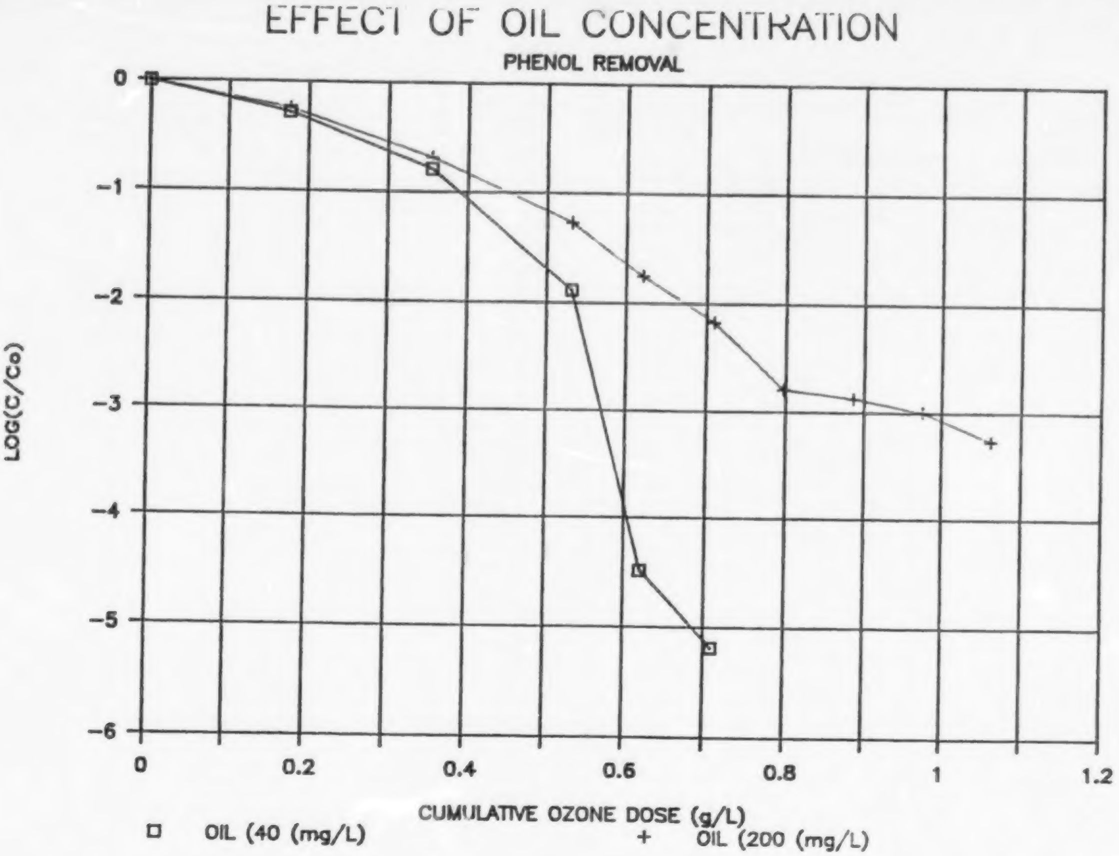
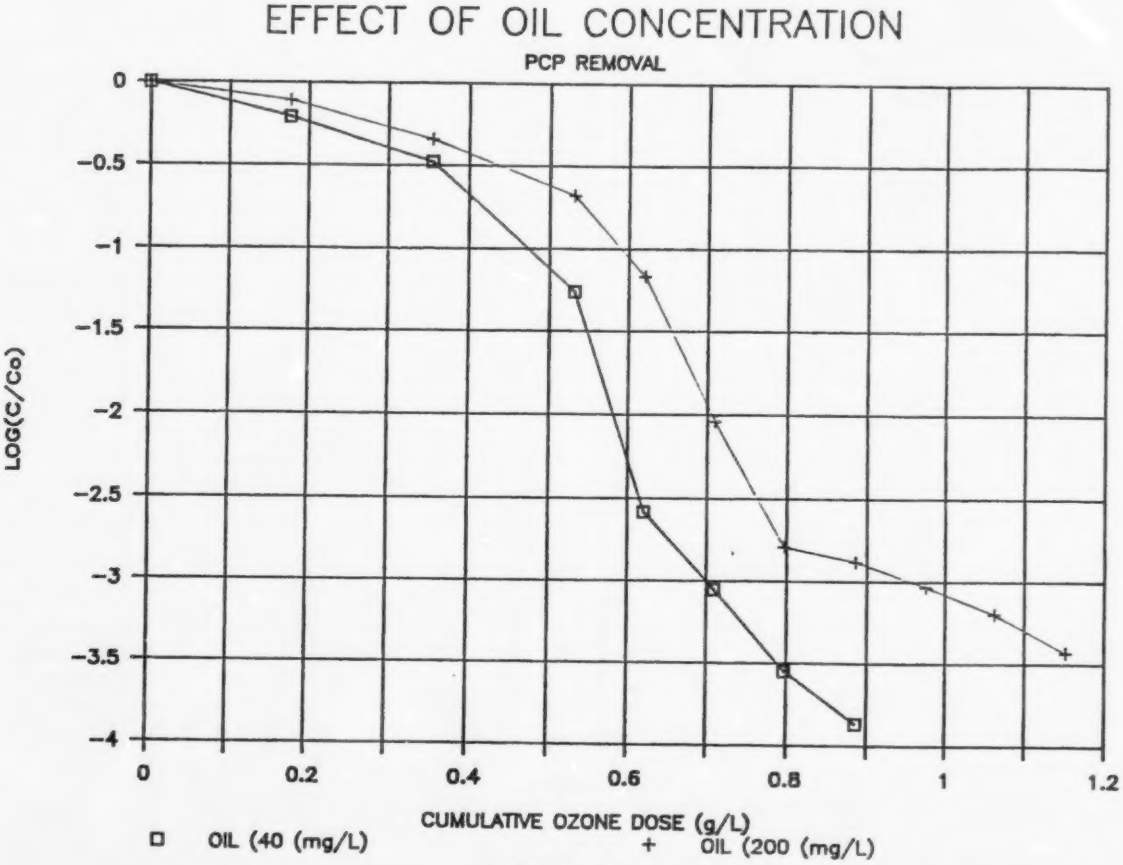


FIGURE 7


EFFECT OF OIL CONCENTRATION

PHENOL REMOVAL

FIGURE 8 (a)


EFFECT OF OIL CONCENTRATION

PCP REMOVAL

FIGURE 8 (b)

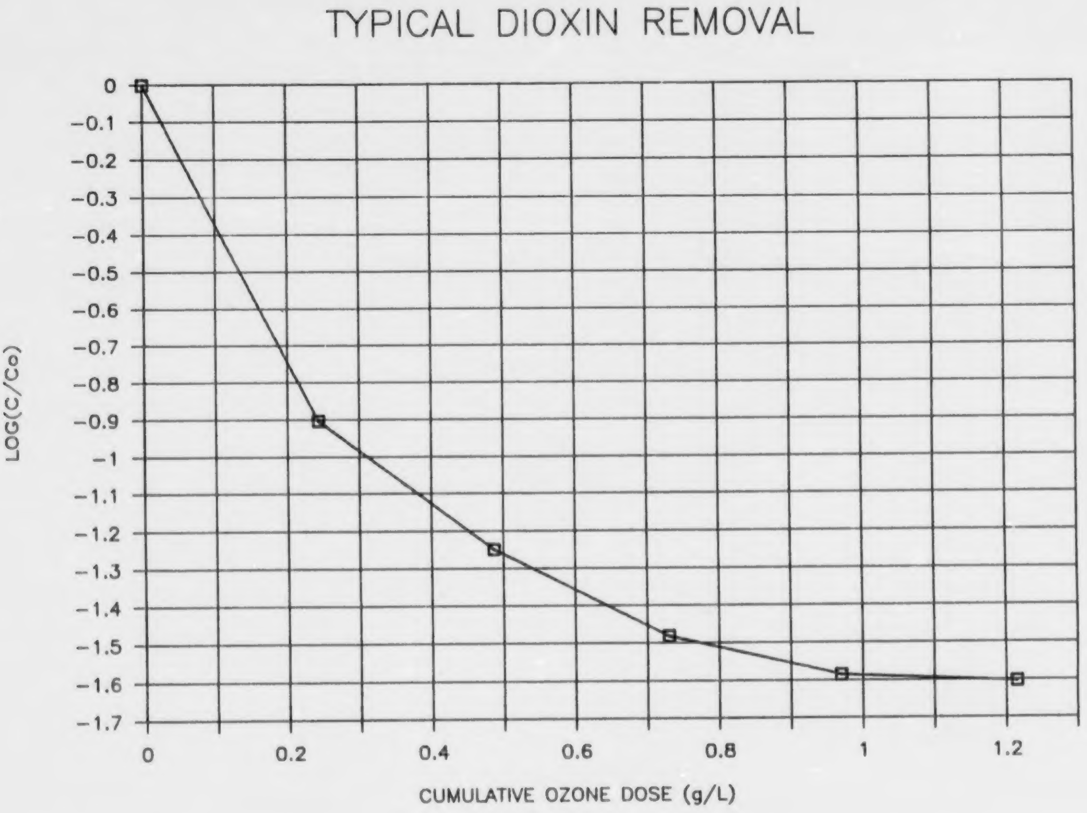


FIGURE 9

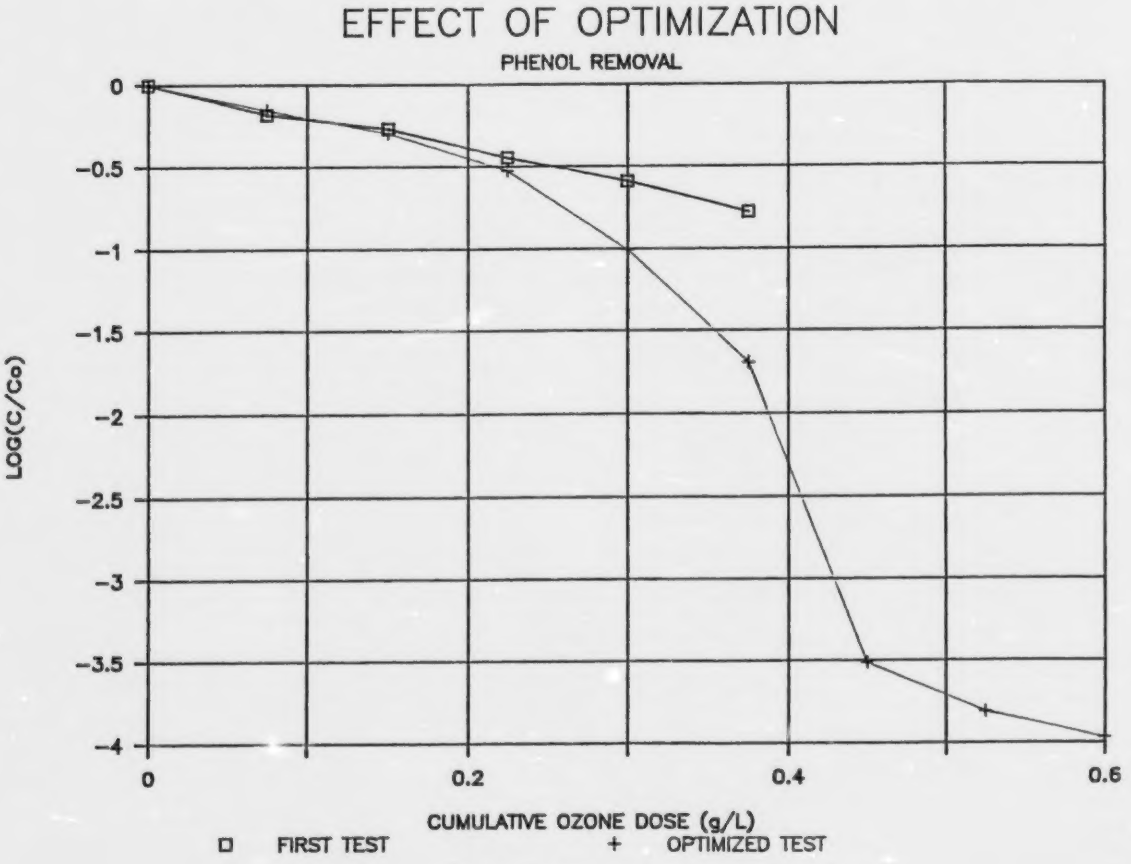


Figure 10

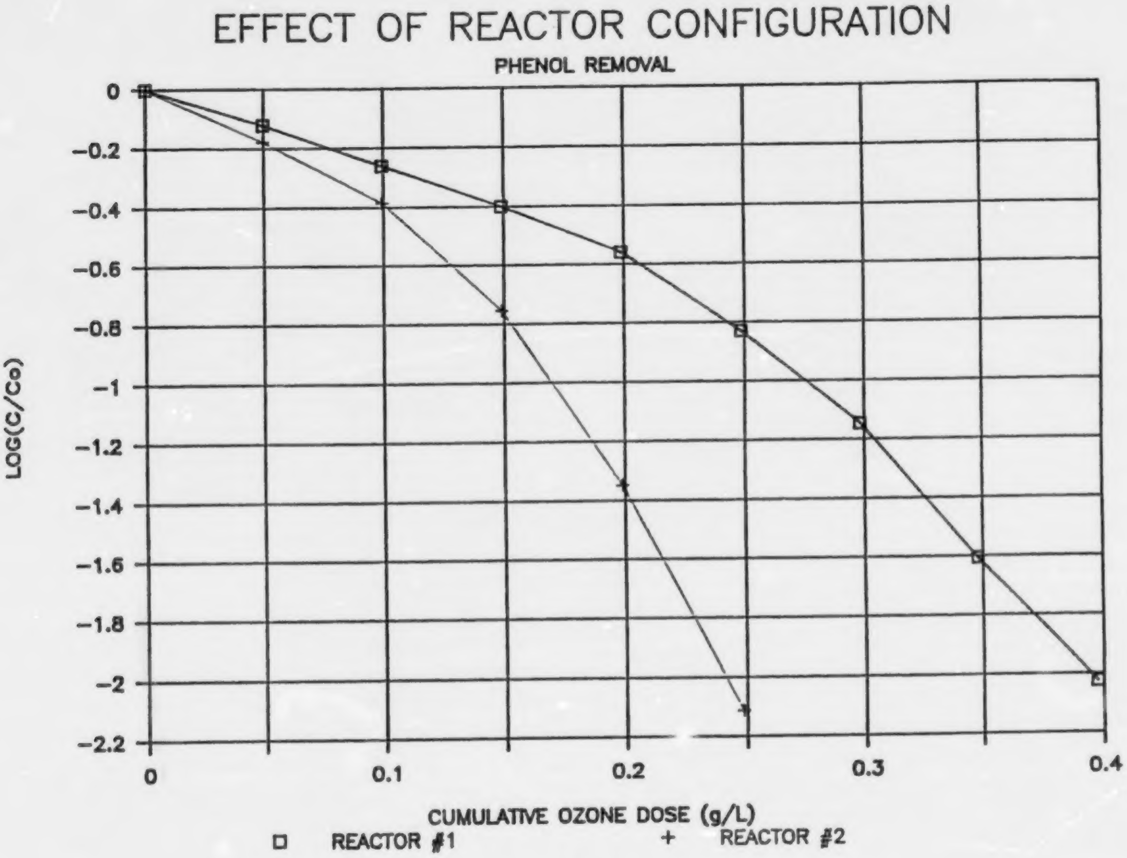

EFFECT OF REACTOR CONFIGURATION
PHENOL REMOVAL
LOG(C/Co)
0
-0.2
-0.4
-0.6
-0.8
-1
-1.2
-1.4
-1.6
-1.8
-2
-2.2
0
0.1
0.2
0.3
0.4
CUMULATIVE OZONE DOSE (g/L)
□ REACTOR #1
+ REACTOR #2

FIGURE 11

DEVELOPMENT OF A SAMPLE CLEAN-UP METHOD FOR SEMI-VOLATILE ORGANICS IN POLYMER BEARING WASTEWATER

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ABSTRACT

The United States Environmental Protection Agency (U.S. EPA) promulgated on 5 November 1987 the effluent limitations guidelines and standards that apply to wastewater discharges from organic chemical, plastics and synthetic fibers (OCPSF) manufacturing plants. The regulations limit the quantities of certain organic and inorganic Priority Pollutants that may be discharged to navigable waters and publicly owned treatment works. The methodology employed for the analysis of the Priority Pollutants in the effluent discharges are applicable to methodologies used for liquids and sludges under the U.S. EPA Hazardous Waste Program (RCRA). The difficulties in analyzing these effluents are not unique to wastewaters and may also be encountered in the analysis of solid waste. A validation study using traditional analytical procedures was conducted on effluent streams from an OCPSF plant to determine whether these protocols can be utilized to ascertain the presence or absence of the organic Priority Pollutants. The traditional methodologies included base-neutral and acid extraction (BNA) for organic compounds followed by gas chromatography/mass spectrometry (GC/MS).

The OCPSF effluents contained polymeric constituents that prohibited the extraction of the BNA fractions. Gel permeation chromatography was investigated as an alternative preparatory technique but was proven to be unsuitable.

A method has been developed for semi-volatile analysis whereby polymeric constituents are precipitated prior to extraction procedures. This preparation method proved successful as adequate surrogate spike recoveries were attained, and accuracy and precision were documented. The manner in which validation was conducted and the results will be presented in this case study.

INTRODUCTION

The U.S. EPA semi-volatile extraction method for wastewater and hazardous waste matrices (EPA method 3510) stipulates extraction of the target compounds by serially extracting with methylene chloride at a pH greater than 11 and again at pH less than 2. While this method is effective for many matrices, it is not amenable to certain OCPSF effluents due to the potential presence of polymeric constituents. Acidification of certain OCPSF effluents containing substantial concentrations of polymeric constituents resulted in precipitation of the polymers and converted the wastewater sample into a solid-gel matrix. In this form, extraction of the target compounds and subsequent analysis were prohibited. In those cases where the effluents were found to contain lower concentrations of polymeric constituents, acidification did not result in a heavy precipitate; however, the presence of the polymers in the extracts prohibited them from being concentrated to the 1-ml volume required for achieving routine method detection limits.

Gel Permeation Chromatography (GPC) has been used to remove high molecular weight interferences from sample extracts prior to gas chromatography analysis. The GPC method has been successfully used to remove lipids from biological samples and may be applied to a variety of environmental samples (EPA method 3640). The OCPSF effluents appeared to be likely candidates. Attempts to separate the polymeric constituents by GPC resulted in column solidification. Dilution of the effluent stream within an analytical working range failed to resolve the solidification problem. Further dilution proved successful; however, the detection limits for the target compounds were elevated beyond the limits set by the OCPSF effluent regulations. Failure of the GPC method is believed to be attributed to molecular weight interference in excess of the method capacity.

THEORY

Our knowledge of the molecular characteristics of the polymers lead us to believe that flocculation of the polymer was a feasible treatment technique. A separation technique based on flocculation of the polymers prior to extraction was developed by WESTON. The flocculation technique subscribes to the traditional wastewater treatment of ferric sulfate and lime. The flocculation reaction may be illustrated as follows:

$$\underset{\text{Ferric Sulfate}}{Fe_2(SO_4)_3} + \underset{\text{Calcium Hydroxide}}{Ca(OH)_2} \longrightarrow \underset{\text{Ferric Hydroxide}}{Fe(OH)_3}$$

Ferric sulfate and calcium hydroxide are added to form the insoluble ferric hydroxide. A gelatinous floc is formed that provides a bonding site for the colloidal macro molecular (polymer) constituents to form weak bonds with the precipitate. Removal of the precipitate, therefore, facilitated removal of the bonded polymeric constituents. A commercially available flocculating agent primarily composed of ferric sulfate was used to treat the OCPSF samples prior to BNA extraction. A validation study was conducted by WESTON to determine to what extent, if any, the ferric sulfate/lime treatment of the effluent affected the analytical results.

STUDY DESIGN

Wastewater samples were collected daily from a thermoplastic resin production facility for 6 days. The samples were collected, preserved and shipped to WESTON in accordance with the U.S. EPA Contract Laboratory protocol (preserved at 4°C).

The validation study was designed to comply with the criteria established for the EPA Contract Laboratory Program (CLP). The quality control parameters included the use of surrogates, matrix and blank spikes, duplicates and blank samples. Data validation was conducted using the CLP Functional Guidelines for Evaluating Organic Analyses.

METHOD

A 1-liter sample was treated by adding 4 ml of 25% ferric sulfate and 36 ml of 10% calcium hydroxide. The sample was shaken manually and the precipitate was allowed to separate and settle. The liquid phase was decanted and extracted by EPA Method 3510 and analyzed by GC/MS EPA Method 8270. Method 8270 is used to quantify BNA compounds soluble in methylene chloride and eluted from a GC fused-silica capillary column coated with a slightly polar silicone. The practical quantitation limit for determining an individual compound is approximately 1-200 mg/kg for wastes (depending on matrix) and 10 ug/l for aqueous samples.

RESULTS AND DISCUSSION

The objectives of the validation study are to determine whether the ferric sulfate treatment is a viable treatment technique and to determine if this treatment affected the analytical results. In order to meet these objectives, (1) quality control samples were subjected to the flocculation treatment to monitor any effects upon the analytical performance under treatment conditions and (2) wastewater samples were treated and analyzed at detection limits below the OCPSF maximum monthly average pretreatment standards listed in Table 1.

Table 2 reports the percent recoveries for the quality control samples. The CLP recovery limits guidelines are included in Tables 2 and 3 to serve as a relative measure of performance. The surrogate and target compounds recoveries were generally within the CLP limits for the blank and matrix spike samples. The outliers were primarily biased to the upper limits. Matrix interference was noted during the analysis of the matrix spike (RFW # 001MS), which accounts for the loss of the 4-nitrophenol and a reduced recovery for the 2-fluorophenol.

Table 3 presents the analytical results of the wastewater samples. Surrogate recoveries in most samples met the CLP recovery criteria. The outliers are believed to be attributable to the relative high background of non-target compounds. The surrogate tribromophenol was not detected in one sample (Sample # 2). This appears to be an isolated case and does not appear to be caused by the ferric sulfate/lime treatment. The detection limits attainable in the validation study for the wastewater samples were well below 100 parts per billion. These levels were unattainable without the ferric sulfate treatment. The recovery data indicate that acceptable performance levels were attained which substantiates that the flocculation technique does not affect the analytical results and that the technique effectively treats certain OCPSF waste streams.

CONCLUSION

Wastewater samples containing high concentrations of polymer and copolymer compounds were successfully prepared for analysis utilizing a clean-up technique involving the chemical precipitation and subsequent removal of these constituents. Target compound concentrations were not impacted by the sample preparation steps as indicated by achieved surrogate and matrix spike recoveries. It appears that the lower molecular sized organic target wastewater constituents did not react or bond with the precipitate, and therefore, largely remained in solution and were not affected by the ferric sulfate/lime sample clean-up technique.

ACKNOWLEDGEMENT

Special thanks are extended to Caroline Power and Pat Sobota for their assistance with graphics.

Table 1

Pretreatment Standards

Effluent Characteristics	Maximum for Any One Day	Maximum for Monthly Average	Method Detection Limit
Acenaphthene	47	19	10
Benzene	134	57	5
Carbon Tetrachloride	380	142	5
Chlorobenzene	380	142	5
1,2,4-Trichlorobenzene	794	196	10
Hexachlorobenzene	794	196	50
1,2-Dichloroethane	574	180	5
1,1,1-Trichloroethane	59	22	5
Hexachloroethane	794	196	10
1,1-Dichloroethane	59	22	5
1,1,2-Trichloroethane	127	32	5
Chloroethane	295	110	10
Chloroform	325	111	5
1,2-Dichlorobenzene	794	196	10
1,3-Dichlorobenzene	380	142	10
1,4-Dichlorobenzene	380	142	10
1,1-Dichloroethylene	60	22	5
1,2-trans-Dichloroethylene	66	25	5
1,2-Dichloropropane	794	196	5
1,3-Dichloropropylene	794	196	5
2,4-Dimethylphenol	47	19	10
Ethylbenzene	380	142	5
Fluoranthene	54	22	10
Methylene Chloride	170	36	5
Methyl Chloride	295	110	10
Hexachlorobutadiene	380	142	10
Naphthalene	47	19	10
Nitrobenzene	6,402	2,237	10
2-Nitrophenol	231	65	10
4-Nitrophenol	576	162	50
4,6-Dinitro-o-cresol	277	78	
Phenol	47	19	10
Bis(2-ethylhexyl)phthalate	258	95	10
Di-n-butyl phthalate	43	20	10
Diethyl phthalate	113	46	10
Dimethyl phthalate	47	19	10
Anthracene	47	19	10
Fluorene	47	19	10
Phenanthrene	47	19	10
Pyrene	48	20	10
Tetrachloroethylene	164	52	10
Toluene	74	28	5
Trichloroethylene	69	26	5
Vinyl chloride	172	97	10
Total Cyanide	1,200	420	10
Total Lead	690	320	5
Total Zinc	2,610	1,050	20

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1 All units are micrograms per liter.

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Table 2

Percent Recoveries for Blank and Matrix Spike Samples

Sample Information MDL = 10xD.F., except (2) = 50x, (3) = 20x							
	Sample#:	BS(187)	BS(194)	BS(206)	BSD(206)	7	
	RFW#:	B.S.	B.S.	B.S.	B.S.D.	001 MS	
	Matrix:	Water	Water	Water	Water	Water	CLP Limits (Percent)
	D.F.:	1	1	1	1	1.3	
	Units:	ug/L	ug/L	ug/L	ug/L	ug/L	
Surrogate Recovery (%)	2-Fluorophenol:	76%	56%	32%	39%	17%	21 - 100
	Phenol-d5:	54%	39%	23%	27%	30%	10 - 94
	2,4,6-Br3-Phenol:	149%	120%	95%	103%	52%	10 - 123
	Nitrobenzene-d5:	92%	84%	62%	60%	84%	35 - 114
	2-Fluorobiphenyl:	98%	82%	70%	72%	110%	43 - 116
	p-Terphenyl-d14:	104%	84%	80%	84%	90%	33 - 141
Phenol		61%	45%	27%	31%	43%	12-89
bis(2-Chloroethyl)Ether		NR	NR	NR	NR	NR	
2-Chlorophenol		121%	97%	61%	71%	66%	27-123
1,3-Dichlorobenzene		10 U	10 U	10 U	10 U	13 U	
1,4-Dichlorobenzene		72%	72%	42%	36%	72%	36-97
1,2-Dichlorobenzene		10 U	10 U	10 U	10 U	13 U	
bis(2-Chloroisopropyl)Ether		10 U	10 U	10 U	10 U	13 U	
N-Nitroso-di-n-propylamine		102%	94%	68%	64%	96%	41-116
Hexachloroethane		10 U	10 U	10 U	10 U	13 U	
Nitrobenzene		10 U	10 U	10 U	10 U	13 U	
Isophorone		10 U	10 U	10 U	10 U	NR	
2-Nitrophenol		10 U	10 U	10 U	10 U	13 U	
2,4-Dimethylphenol		10 U	10 U	10 U	10 U	13 U	
bis(2-Chloroethoxy)Methane		NR	NR	NR	NR	NR	
2,4-Dichlorophenol		10 U	10 U	10 U	10 U	13 U	
1,2,4-Trichlorobenzene		76%	74%	48%	48%	86%	39-98
Naphthalene		10 U	10 U	10 U	10 U	13 U	
Hexachlororbutadiene		10 U	10 U	10 U	10 U	13 U	
4-Chloro-3-methylphenol		126%	103%	72%	82%	63%	23-97
Hexachlorocyclopentadiene		NR	NR	NR	NR	NR	
2,4,6-Trichlorophenol		NR	NR	NR	NR	NR	
2-Chloronaphthalene		NR	NR	NR	NR	NR	
Dimethyl Phthalate		10 U	10 U	10 U	10 U	13 U	
Acenaphthylene		10 U	10 U	10 U	10 U	13 U	
Acenaphthene		100%	94%	76%	76%	102%	46-118
2,4-Dinitrophenol(2)		50 U	50 U	50 U	50 U	65 U	
4-Nitrophenol(2)		60%	42%	25%	28%	ND	10-80
2,4-Dinitrotoluene		108%	90%	80%	80%	60%	24-96
2,6-Dinotrotoluene		10 U	10 U	10 U	10 U	13 U	

(1) = Cannot be distinguished from diphenylan.
NR = Not detected.
J = Present at less than detection limit.
B = Present in blank.
NR = Not requested.
MS = Matrix Spike.
BS = Blank Spike.
BSD = Blank Spike Duplicate.
U = Not detected at the minimum detection limit for the sample.
1283E

Table 2
(continued)

Sample Information MDL = 10xD.F., except (2) = 50x, (3) = 20x	Sample#:	BS(187)	BS(194)	BS(206)	BSD(206)	7	CLP Limits (Percent)
	RFW#:	B.S.	B.S.	B.S.	B.S.D.	001 MS	
	Matrix:	Water	Water	Water	Water	Water	
	D.F.:	1	1	1	1	1.3	
	Units:	ug/L	ug/L	ug/L	ug/L	ug/L	
Surrogate Recovery (%)	2-Fluorophenol:	76%	56%	32%	39%	17%	21 - 100
	Phenol-d5:	54%	39%	23%	27%	30%	10 - 94
	2,4,6-Br3-Phenol:	149%	120%	95%	103%	52%	10 - 123
	Nitrobenzene-d5:	92%	84%	62%	60%	84%	35 - 114
	2-Fluorobiphenyl:	98%	82%	70%	72%	110%	43 - 116
	p-Terphenyl-d14:	104%	84%	80%	84%	90%	33 - 141
Diethyl Phthalate		10 U	10 U	10 U	10 U	10 J	
4-Chlorophenyl-phenylether		NR	NR	NR	NR	NR	
Fluorene		10 U	10 U	10 U	10 U	13 U	
4,6-Dinitro-2-methylphenol(2)		NR	NR	NR	NR	13 U	
N-Nitrosodiphenylamine(1)		NR	NR	NR	NR	NR	
4-Bromophenyl-phenylether		NR	NR	NR	NR	NR	
Hexachlorobenzene		10 U	10 U	10 U	10 U	13 U	
Pentachlorophenol(2)		139%	93%	56%	55%	57%	9-103
Phenanthrene		10 U	10 U	10 U	10 U	13 U	
Anthracene		10 U	10 U	10 U	10 U	13 U	
di-n-Butyl Phthalate		2 JB	10 U	10 U	10 U	13 U	
Fluoranthene		10 U	10 U	10 U	10 U	13 U	
Pyrene		110%	88%	86%	88%	100%	26-127
Butyl Benzyl Phthalate		NR	NR	NR	NR	NR	
3,3'-Dichlorobenzidine(3)		NR	NR	NR	NR	NR	
Benzo(a)Anthracene		10 U	10 U	10 U	10 U	13 U	
bis(2-Ethylhexyl)Phthalate		8 JB	19 B	10 U	10 U	55 B	
Chrysene		10 U	10 U	10 U	10 U	13 U	
di-n-Octyl Phthalate		NR	NR	NR	NR	NR	
Benzo(b)Fluoranthene		10 U	10 U	10 U	10 U	13 U	
Benzo(k)Fluoranthene		10 U	10 U	10 U	10 U	13 U	
Benzo(a)Pyrene		10 U	10 U	10 U	10 U	13 U	
Indeno(1,2,3-cd)Pyrene		NR	NR	NR	NR	NR	
Dibenz(a,h)Anthracene		NR	NR	NR	NR	NR	
Benzo(g,h,i)Perylene		NR	NR	NR	NR	NR	
N-Nitrosodiemethylamine		NR	NR	NR	NR	NR	
Benzidine		NR	NR	NR	NR	NR	

(1) = Cannot be distinguished from diphenylan.
NR = Not detected.
J = Present at less than detection limit.
B = Present in blank.
NR = Not requested.
MS = Matrix Spike.
BS = Blank Spike.
BSD = Blank Spike Duplicate.
U = Not detected at the minimum detection limit for the sample.
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Table 3

Wastewater Analytical Results

Sample Information MDL = 10xD.F., except (2) = 50x, (3) = 20x	Sample#:	BLANK (187)	1	2	3	4	5	BLANK(194)	6	
	RFW#:	Blank	001	001	017	001	018	Blank	001	
	Matrix:	Water	Water	Water	Water	Water	Water	Water	Water	CLP
	D.F.:	1	1	1.4	1	1.5	1	1	1	Limits
	Units:	ug/L	ug/L	ug/L	ug/L	ug/L	ug/L	ug/L	ug/L	(Percent)
Surrogate Recovery (%)	2-Fluorophenol:	49%	35%	38%	36%	53%	28%	50%	47%	21 - 100
	Phenol-d5:	31%	29%	33%	35%	37%	23%	34%	39%	10 - 94
	2,4,6-Br3-Phenol:	79%	53%	ND	118%	66%	101%	16%	38%	10 - 123
	Nitrobenzene-d5:	92%	108%	85%	132%	80%	96%	106%	78%	35 - 114
	2-Fluorobiphenyl:	88%	82%	87%	100%	74%	100%	96%	88%	43 - 116
	p-Terphenyl-d14:	106%	96%	78%	112%	68%	110%	106%	100%	33 - 141
Phenol		10 U	12	14 U	5 J	68	10 U	10 U	16 U	
2-Chlorophenol		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
1,3-Dichlorobenzene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
1,4-Dichlorobenzene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
1,2-Dichlorobenzene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
bis(2-Chloroisopropyl)Ether		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Hexachloroethane		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Nitrobenzene		10 U	10 U	14 U	10 U	27	10 U	10 U	16 U	
2-Nitrophenol		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
2,4-Dimethylphenol		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
2,4-Dichlorophenol		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
1,2,4-Trichlorobenzene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Naphthalene		10 U	10 U	14 U	10 U	36	7 J	10 U	16 U	
Hexachlororbutadiene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
4-Chloro-3-methylphenol		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Dimethyl Phthalate		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Acenaphthylene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	

ND = Not detected.
J = Present at less than detection limit.
B = Present in blank.
U = Not detected at the minimum detection limit for the sample.

Table 3
(continued)

Sample Information MDL = 10xD.F., except (2) = 50x, (3) = 20x	Sample#:	BLANK (187)	1	2	3	4	5	BLANK(194)	6	CLP Limits (Percent)
	RFW#:	Blank	001	001	017	001	018	Blank	001	
	Matrix:	Water	Water	Water	Water	Water	Water	Water	Water	
	D.F.:	1	1	1.4	1	1.5	1	1	1	
	Units:	ug/L	ug/L	ug/L	ug/L	ug/L	ug/L	ug/L	ug/L	
Acenaphthene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
2,4-Dinitrotoluene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
2,6-Dinitrotoluene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Diethyl Phthalate		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Fluorene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Hexachlorobenzene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Phenanthrene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Anthracene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
di-n-Butyl Phthalate		2 J	2JB	3JB	25 B	15 U	10 U	10 U	16 U	
Fluoranthene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Pyrene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Benzo(a)Anthracene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
bis(2-Ethylhexyl)Phthalate		4 J	3JB	15 B	10 U	15 U	10 U	40	54 B	
Chrysene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Benzo(b)Fluoranthene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Benzo(k)Fluoranthene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
Benzo(a)Pyrene		10 U	10 U	14 U	10 U	15 U	10 U	10 U	16 U	
2,4-Dinitrophenol (2)		50 U	50 U	70 U	50 U	75 U	50 U	50 U	80 U	
4-Nitrophenol (2)		50 U	50 U	70 U	50 U	75 U	50 U	50 U	80 U	

ND = Not detected.
J = Present at less than detection limit.
B = Present in blank.
U = Not detected at the minimum detection limit for the sample.

Table 3
(continued)

Sample Information MDL = 10xD.F., except (2) = 50x, (3) = 20x	Sample#: RFW#: Matrix: D.F.: Units:	BLANK (194) BLANK Water 1 ug/L	7 001 Water 1.5 ug/L	BLANK(206) BLANK Water 1 ug/L	8 001 Water 1 ug/L	CLP Limits (Percent)
Surrogate Recovery (%)	2-Fluorophenol:	50%	23%	30%	19%	21 - 100
	Phenol-d5:	34%	38%	20%	23%	10 - 94
	2,4,6-Br3-Phenol:	16%	111%	60%	12%	10 - 123
	Nitrobenzene-d5:	106%	84%	38%	20%	35 - 114
	2-Fluorobiphenyl:	96%	84%	52%	4%	43 - 116
	p-Terphenyl-d14:	106%	72%	72%	20%	33 - 141
Phenol		10 U	15 U	10 U	10 U	
2-Chlorophenol		10 U	15 U	10 U	10 U	
1,3-Dichlorobenzene		10 U	15 U	10 U	10 U	
1,4-Dichlorobenzene		10 U	15 U	10 U	10 U	
1,2-Dichlorobenzene		10 U	15 U	10 U	10 U	
bis(2-Chloroisopropyl)Ether		10 U	15 U	10 U	10 U	
Hexachloroethane		10 U	15 U	10 U	10 U	
Nitrobenzene		10 U	15 U	10 U	10 U	
2-Nitrophenol		10 U	15 U	10 U	10 U	
2,4-Dimethylphenol		10 U	15 U	10 U	10 U	
2,4-Dichlorophenol		10 U	15 U	10 U	10 U	
1,2,4-Trichlorobenzene		10 U	15 U	10 U	10 U	
Naphthalene		10 U	15 U	10 U	10 U	
Hexachlororbutadiene		10 U	15 U	10 U	10 U	
4-Chloro-3-methylphenol		10 U	15 U	10 U	10 U	
Dimethyl Phthalate		10 U	15 U	10 U	68	
Acenaphthylene		10 U	15 U	10 U	10 U	

ND = Not detected.
J = Present at less than detection limit.
B = Present in blank.
U = Not detected at the minimum detection limit for the sample.

Table 3
(continued)

Sample Information MDL = 10xD.F., except (2) = 50x, (3) = 20x	Sample#: RFW#: Matrix: D.F.: Units:	BLANK (194) BLANK Water 1 ug/L	7 001 Water 1.5 ug/L	BLANK(206) BLANK Water 1 ug/L	8 001 Water 1 ug/L	CLP Limits (Percent)
Acenaphthene		10 U	15 U	10 U	10 U	
2,4-Dinitrotoluene		10 U	15 U	10 U	10 U	
2,6-Dinitrotoluene		10 U	15 U	10 U	10 U	
Diethyl Phthalate		10 U	12 J	10 U	10 U	
Fluorene		10 U	15 U	10 U	10 U	
Hexachlorobenzene		10 U	15 U	10 U	10 U	
Phenanthrene		10 U	15 U	10 U	10 U	
Anthracene		10 U	15 U	10 U	10 U	
di-n-Butyl Phthalate		10 U	15 U	10 U	10 U	
Fluoranthene		10 U	15 U	10 U	10 U	
Pyrene		10 U	15 U	10 U	10 U	
Benzo(a)Anthracene		10 U	15 U	10 U	10 U	
bis(2-Ethylhexyl)Phthalate		40	50 B	10 U	10 U	
Chrysene		10 U	15 U	10 U	10 U	
Benzo(b)Fluoranthene		10 U	15 U	10 U	10 U	
Benzo(k)Fluoranthene		10 U	15 U	10 U	10 U	
Benzo(a)Pyrene		10 U	15 U	10 U	10 U	
2,4-Dinitrophenol (2)		50 U	75 U	50 U	50 U	
4-Nitrophenol (2)		50 U	75 U	50 U	50 U	

ND = Not detected.
J = Present at less than detection limit.
B = Present in blank.
U = Not detected at the minimum detection limit for the sample.

UTILISATION DE L'OZONE POUR LE TRAITEMENT DES EAUX USÉES DANS UN RÉACTEUR SÉQUENTIEL CHIMIQUE

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Le réacteur séquentiel chimique (R.S.C.) est un procédé unitaire capable de traiter les eaux usées en oxydant la matière organique avec l'ozone. Ce réacteur est alimenté en eaux usées de manière discontinue. Pendant le traitement, des phases acides éliminent le CO_2 et des phases basiques transforment la matière organique en CO_2. Ces phases acides-bases sont effectuées de façon cycliques pour augmenter le rendement du R.S.C..

L'idée de développer le R.S.C. repose sur l'analyse des problèmes de fonctionnement rencontrés régulièrements avec des procédés de boues activés. Les trois principaux problèmes cernés sont les variations dans la charge massique ou volumique à l'entrée des usines, l'apparition aléatoire dans les affluants de produits toxiques pour les microorganismes et les eaux usées carencées. Par conséquent, le R.S.C., basé sur une réaction chimique insensible aux problèmes des organismes vivants, devrait résoudre les principales difficultés de fonctionnement des boues activées.

Avant la présentation des résultats expérimentaux, l'aspect théorique du principe de fonctionnement du R.S.C. sera exposé.

Tous les essais avec le prototype de R.S.C., d'une capacité de 7,5 L, sont réalisés sur une eau usée synthétique. Les charges massiques de cette eau ont été maintenues constantes pour une meilleure interprétation des résultats.

Les résultats ainsi obtenus ont permis de quantifier l'amélioration apportée, sur l'élimination de la D.C.O. (demande chimique en oxygène) et du C.O.T. (carbone organique total), par rapport à une ozonisation traditionnelle à pH constant. D'autre part, le nombre de cycles acide-base, la durée des phases acides et basiques et la durée total du traitement sont établis pour ce type d'eau. Ainsi, pour une D.C.O. de 170 mg/L-O_2 et une concentration en C.O.T. de 62 mg/L-C, l'alternance des phases acides-bases pendant 8 heures produit une diminution de la D.C.O. d'environ 85 % et de 70 % en ce qui concerne le C.O.T..

La conclusion portera sur les avantages d'un tel procédé, les limites de l'étude ainsi que les futures recherches envisagées concernant le R.S.C..

TRAITEMENT COMBINÉ DES EAUX DE LIXIVIATION DE LIEU D'ENFOUISSEMENT SANITAIRE ET D'EAUX USÉES DOMESTIQUES PAR UN SYSTÈME DE BOUES ACTIVÉES

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Texte du résumé

Les eaux de lixiviation d'un lieu d'enfouissement sanitaire représentent la menace la plus sérieuse que fait peser ce mode de gestion des déchets sur la qualité de l'environnement.

Les caractéristiques du lixiviat évoluent avec la stabilisation du site d'enfouissement, ce qui oblige le gestionnaire du site à modifier le système de traitement. Les procédés biologiques présentent un bon potentiel de traitement pour un lixiviat provenant d'un site âgé de quelques années, à cause du rapport DBO_5/DCO encore élevé. Sa très forte charge organique ainsi que sa toxicité, dûe aux métaux et à certaines substances organiques, compromet cependant la performance du système. L'évaluation des constantes de cinétique de réaction Y, k_d et k dans les essais antérieurs de traitement de lixiviat brut a fourni des valeurs inférieures à celles rencontrées dans le traitement des eaux usées domestiques: seule la constante de demi-vitesse K_s augmente. Celà indiquerait une inhibition de la croissance des microorganismes.

Un mode de traitement simple et économique consiste à diluer le lixiviat avec un affluent municipal pour le traiter par un système de boues activées. Nous avons simulé en laboratoire ce procédé avec un réacteur à mélange complet à opération continue d'une capacité d'environ onze litres, à une température de 20°C. Nous avons déversé à l'affluent 5% en volume de lixiviat: la DCO du lixiviat représentait 230 des 357 mg/l de la DCO moyenne. Nous avons obtenu un taux d'enlèvement de la DCO de 90%. La défloculation des boues a fait grimper les pertes à l'effluent jusqu'à 171 mgSS/l: après une acclimatation des microorganismes, les solides en suspension à l'effluent sont redescendus à 12,9 mgSS/l. Les constantes de cinétique de réaction ont toutes des valeurs inférieures à celles utilisées pour le traitement des eaux usées domestiques.

LA REHABILITATION DU RESEAU D'EGOUTS DE FABREVILLE

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Ville de Laval a été appelée, dans le cadre du programme d'assainissement des eaux du Québec, à planifier la réhabilitation de son réseau d'égouts dans le but de réduire les eaux parasites véhiculées. La présente affiche démontre à l'aide de schémas, graphiques et photos, que la réhabilitation est une alternative possible et souvent rentable permettant le fonctionnement adéquat des ouvrages d'assainissement en vue de rencontrer les objectifs environnementaux fixés.

ASPIRECO: UN NOUVEAU CONCEPT POUR L'AERATION DES ETANGS FACULTATIFS

Thierry Freire
ECO EQUIPEMENT FEP INC

PROBLEMATIQUE

Le traitement des eaux usées dans les étangs aérés facultatifs pose parfois des problèmes dus aux surcharges organiques temporaires ou saisonnières associées aux eaux brutes ou aux boues accumulées en tête des cellules. Les phénomènes de court-circuitage hydraulique typique aux grands bassins affectent également de façon parfois importante leur rendement épuratoire des étangs aérés facultatifs.

SOLUTION

Dans le but d'améliorer le procédé, la compagnie Eco Equipement FEP Inc. a développé un nouveau concept pour l'aération des étangs facultatifs: le système ASPIRECO. Ce système est basé sur l'utilisation de jets auto-aspirants permettant d'optimiser la balance des capacités d'oxygénation et de mélange. Les jets sont disposés dans les étangs de façon à générer une circulation interne de la masse liquide favorisant le mélange intégral du contenu des cellules. Ce système permet d'éliminer les courts-circuitages en plus d'assurer la répartition des charges organiques à travers les étangs.

DEVELOPPEMENT

Le développement du jet auto-aspirant a fait l'objet de travaux de recherche réalisés notamment au laboratoire d'hydraulique de l'Ecole Polytechnique de Montréal.

Physique

Ces travaux de recherche ont permis de déterminer, à l'aide d'un modèle grandeur nature, la géométrie optimale du jet de même que ses caractéristiques physiques et hydrauliques. Les résultats ont permi de déterminer une relation mathématique, comprenant les principales dimensions géométriques et la position, qui permet d'évaluer avec suffisamment de précision la quantité d'air aspirée par un jet donné.

Hydraulique

La circulation de l'eau générée par les jets auto-aspirants à l'intérieur d'une cellule a été évaluée à l'aide d'un modèle réduit d'un étang aéré muni de deux rampes d'aération comportant chacune 8 jets auto-aspirants. Un dosage approprié de l'énergie injectée via les jets a laissé voir une circulation générale de la masse d'eau en fossé d'oxydation sur toute la surface de l'étang ce qui a pour effet d'éliminer le court-circuitage hydraulique.

Structural

Le développement des matériaux à utiliser pour la construction des jets et des rampes d'aération de même que l'étude de la forme du moule a fait l'objet d'une étude très poussée par des spécialistes reconnus dans le domaine des matériaux composites. L'analyse structurale des rampes d'aération a été réalisée par le département de génie mécanique de l'Ecole Polytechnique de Montréal et comprend l'évaluation des forces de flottaison, de la déflection des rampes et de la pression admissible que les glaces peuvent exercer sur les conduites.

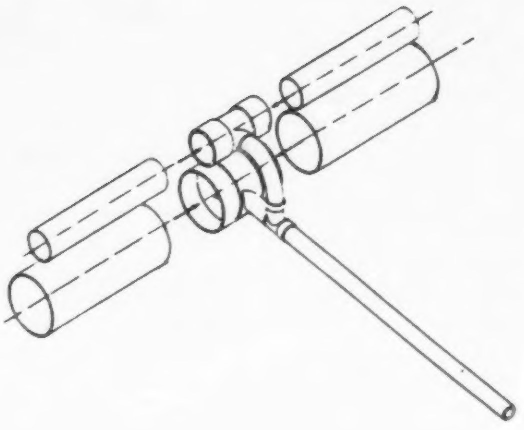

Figure 1.
Assemblage d'un Jet sur rampe d'aération

CAPACITE D'OXYGENATION

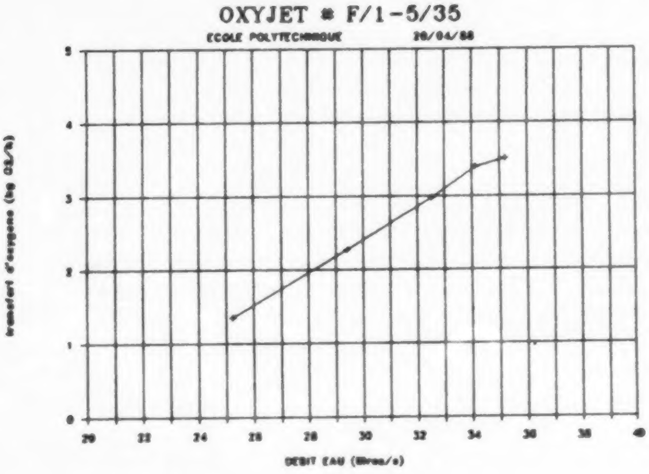

Figure 2.
Oxygène transféré en fonction du débit d'eau pour un Jet.

Les essais d'oxygénation aux conditions standards ont été effectués strictement selon les normes de l'ASCE (1984). Ces essais ont permi d'évaluer l'efficacité énergétique des jets auto-aspirants. Combinés à des pompes submersibles adéquates, les jets à aspiration d'air permettent d'obtenir des efficacités globales de l'ordre de 0,9 à 1,34 Kg O_2/ KW.hre (1,5 à 2,0 lbs O_2/HP.hre). Ces performances permettent de comparer favorablement ces jets à tous les autres dispositifs d'aération rencontrés dans les étangs aérés facultatifs.

CONCLUSION

Le système Aspireco décrit plus haut est d'une innovation majeure dans le principe du procédé de traitement d'eau par étang facultatif. En effet, ce système permet:

- Une circulation de la masse d'eau,
- Une répartition rapide des surcharges,
- Une excellente capacité d'oxygénation,
- Une utilisation plus complète de l'énergie, la chaleur dégagée par les pompes étant injectée dans l'eau du procédé.

Avec l'appui de plusieurs firmes d'ingénieurs-conseils, le système Aspireco est prêt à se tailler une place parmi les plus grands noms dans le domaine de l'aération.

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